



**DEPARTMENT OF HEALTH CARE ACCESS AND INFORMATION
OFFICE OF STATEWIDE HOSPITAL PLANNING AND DEVELOPMENT**

APPLICATION FOR PREAPPROVED PREFABRICATED COMPONENTS AND SYSTEMS

OFFICE USE ONLY

APPLICATION #: PCS- 0004

HCAI Preapproved Prefabricated Components and Systems (PCS)

Type: New Renewal

Manufacturer Information

Manufacturer: DuraFuse Frames connections are fabricated by the steel fabricator that is approved for the project

Manufacturer's Technical Representative: N/A as it varies

Mailing Address: N/A as it varies

Telephone: N/A as it varies Email: N/A as it varies

Product Information

Product Name: DuraFuse

Product Type: Steel Special Moment Frame Connection

General Description: A proprietary special moment frame connection that incorporates a fuse plate to prevent beam and column damage during severe earthquake shaking.

Applicant Information

Applicant Company Name: DuraFuse Frames, LLC

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I hereby agree to reimburse the Department of Health Care Access and Information review fees in accordance with the 2022 California Administrative Code.

Signature of Applicant:  Date: 1/15/2025

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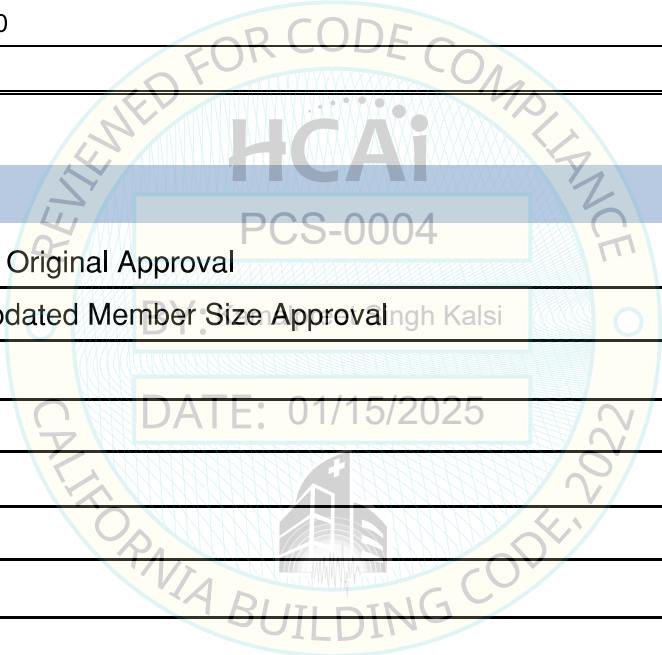
Disciplines Involved

- Structural
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 Mechanical
 Electrical
 Plumbing
 Fire Life Safety

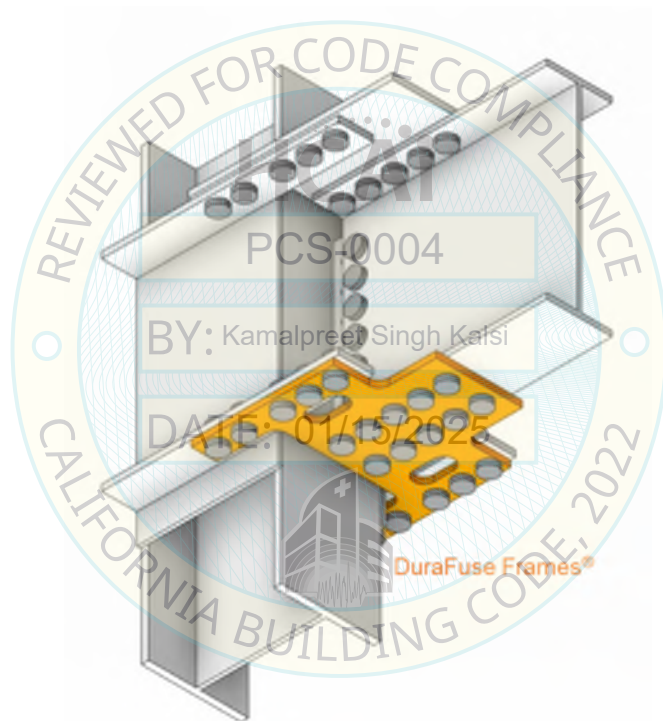
OFFICE USE ONLY – HCAI APPROVAL	
Signature: <u><i>Kamalpreet Singh Kalsi</i></u>	Date: <u>1/15/2025</u>
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Approved Version Number <u>V2.0</u>	

Version History

- V1.0 September 18, 2023 - Original Approval
- V2.0 January 15, 2025 - Updated Member Size Approval gh Kalsi
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DuraFuse Frames[®] (DFF) HCAI Plan Review Guide



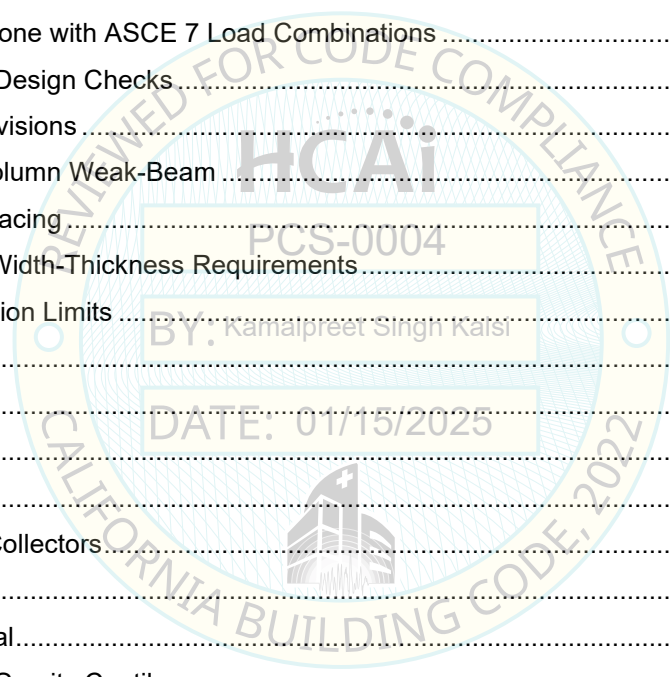
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Version: 2.0



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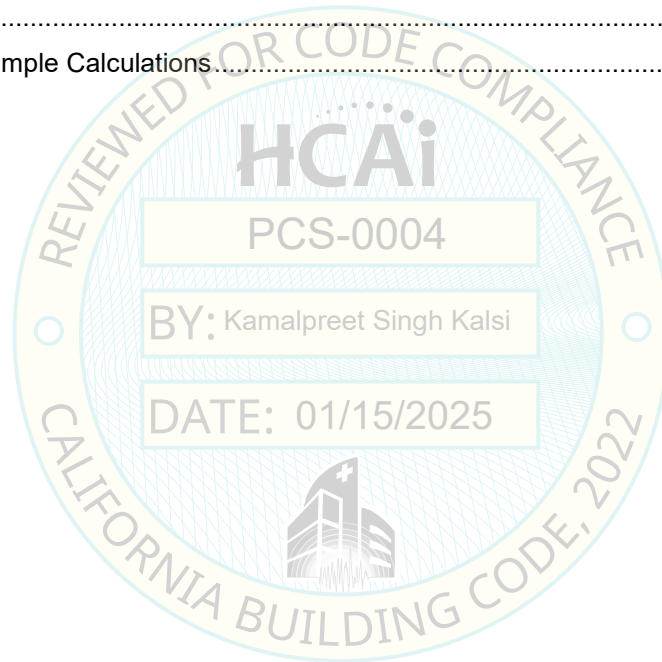
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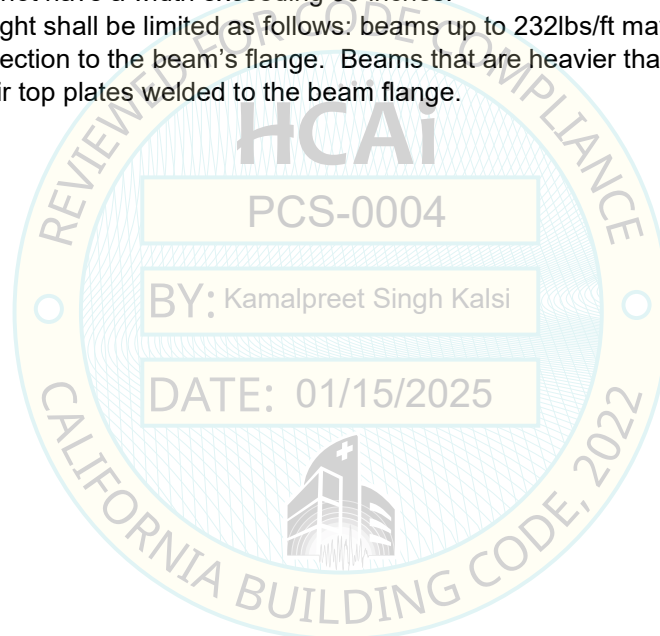
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1 Limits of Pre-Approval

The DuraFuse Frames (DFF) connection in accordance with 2022 California Building Code (with HCAI amendments) and AISC 358-22 Chapter 15, shall be permitted under the following additional conditions:

- a. A linear analysis procedure shall be used for design of the SMF systems using the DFF when permitted in accordance with ASCE 7.
- b. The biaxial dual-strong axis and column minor axis configurations of the moment connection shall be considered as an alternative system.
- c. DFF connection bolts shall not slip under wind design demand loads. The connection shall be designed to prevent slip using AISC 360 Equation J3-4, where the slip resistance is taken to be 0.3. Story drift limitations per 2022 CBC § 1609A.1.2 shall also be met.
- d. Beam flange width-to-thickness ratio shall not exceed λ_p of $0.38 \cdot \sqrt{E/F_y}$ per AISC 360 Table B4.1b Case 11.
- e. Rolled shape column depth shall be limited to W40x maximum. The Column depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes. Built-up box-columns shall not have a width exceeding 30 inches.
- f. The beam weight shall be limited as follows: beams up to 232lbs/ft may have a bolted or welded top plate connection to the beam's flange. Beams that are heavier than 232lbs/ft up to 397lbs/ft shall have their top plates welded to the beam flange.



2 Connection Description

2.1 Overview

The DuraFuse Frames (DFF) connection is prequalified in AISC 358-22 for the use in steel special moment frames (SMF). The DFF connection utilizes plates to connect beams to columns. For I-shaped columns, the column has cover plates on each side that are fillet welded to the column flanges [Fig. 1(a) and (b)]. For box or HSS columns, the sides of the column may function as the cover plates [Fig. 1(c)]. Four external continuity plates that extend past the face of the column are welded to the column cover plates. The column has a shear tab with horizontal slotted holes, that is welded to the column face. The beam web, with standard holes, is attached to the shear tab with pretensioned bolts. The beam flanges are attached to the external continuity plates via top plates and a fuse plate (Fig. 1). The beam flanges and external continuity plates have standard holes, while the top plates and fuse plate have short-slotted or oversized holes. When the top plates are welded to the beam flange, the corresponding, previously referenced, bolt holes are omitted. The fuse plate is proportioned such that certain regions of the plate experience shear yielding when the connection is subjected to severe earthquake loading (Fig. 2). The fuse plate is bolted in place, so that it could be removed and replaced following a severe earthquake. The top plates are intended to experience minimal yielding, such that they would not require repair following a severe earthquake. The various plates in the connection are proportioned such that the beam and column remain essentially elastic.

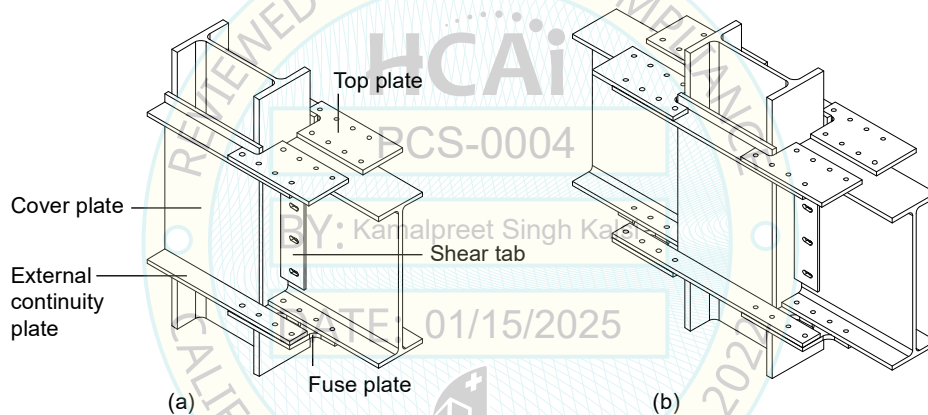


Fig. 1. DuraFuse Frames (DFF) connections (a) one-sided with wide-flange beam and column (b) two-sided with wide flange beams and column.

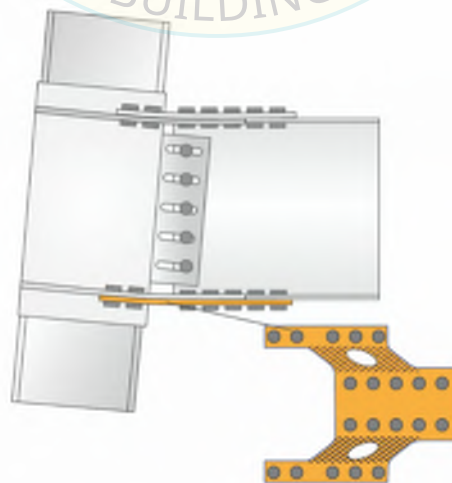


Fig. 2 Inelastic response of a DuraFuse connection with yielding confined to the fuse plate.

2.2 Details and Nomenclature

Figures 3 through 7 define the symbols for various plate thicknesses, dimensions, weld sizes, and bolt quantities that are pertinent to the connection and discussed in the design guide.

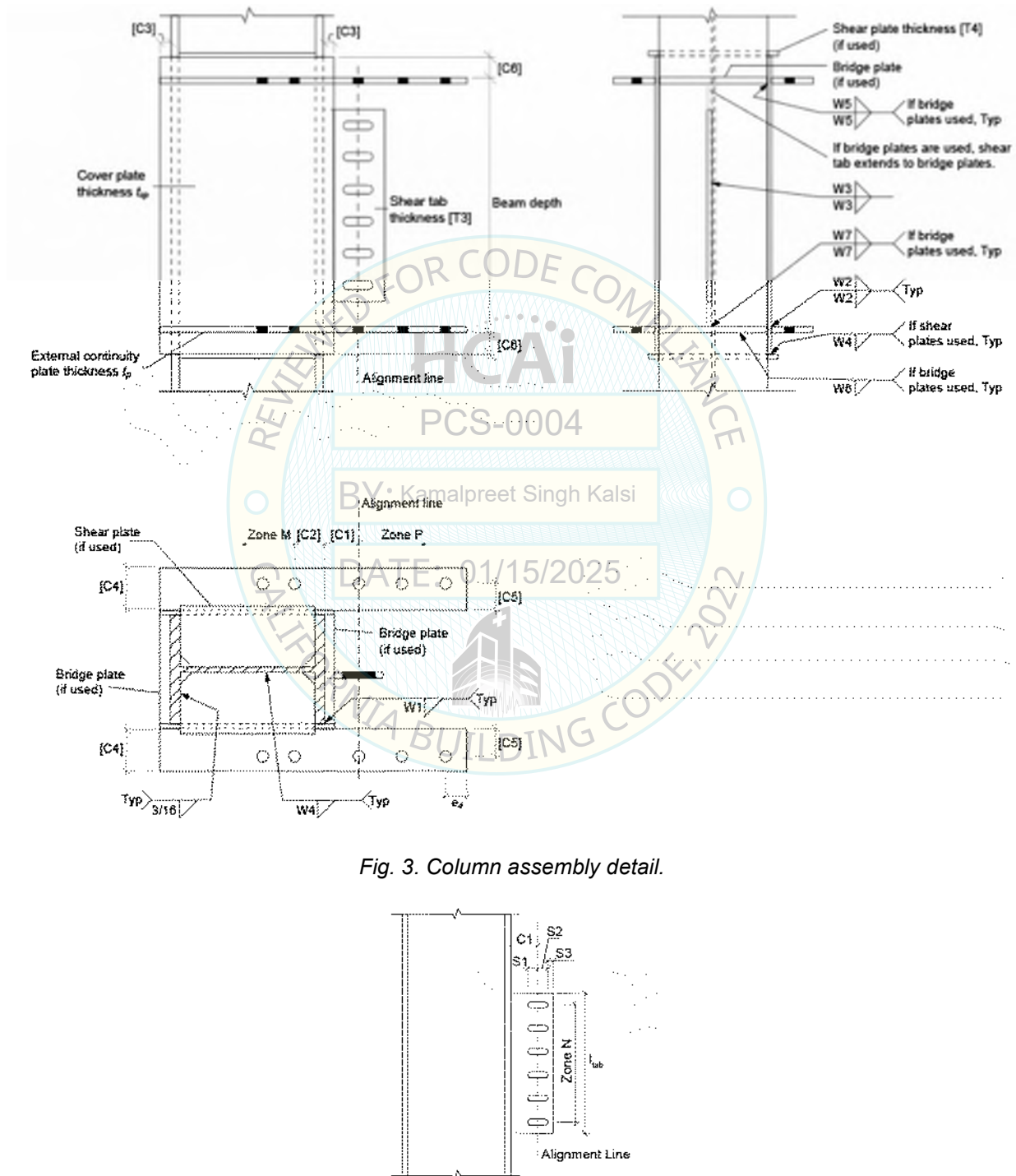


Fig. 3. Column assembly detail.

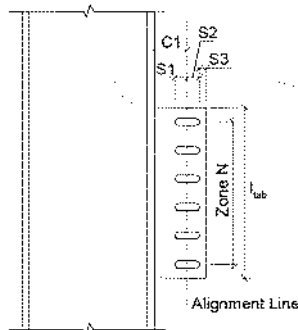
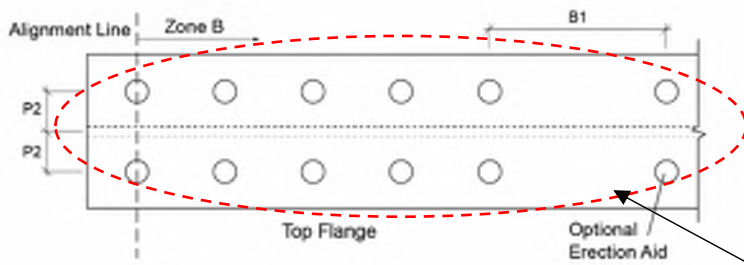


Fig. 4. Shear tab detail.



NOTE: When top plates are welded bolt holes here are omitted

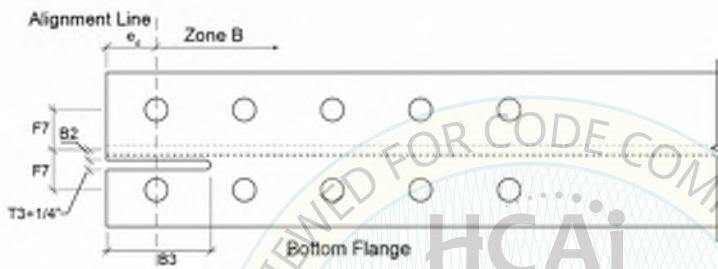


Fig. 5. Beam end detail.

BY: Kamalpreet Singh Kalsi

DATE: 01/15/2025

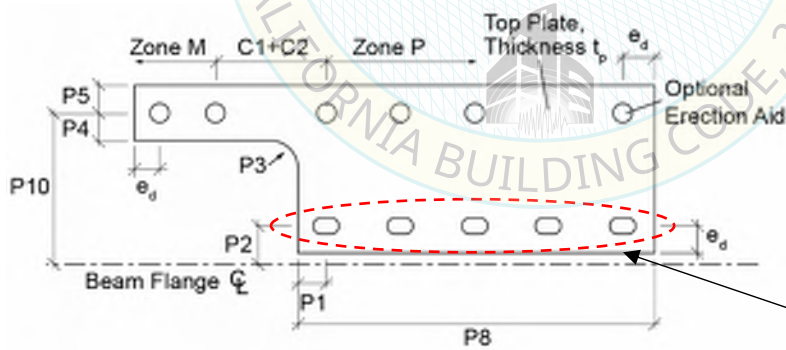


Fig. 6. Top plate detail.

NOTE: When top plates are welded short-slotted bolt here are omitted

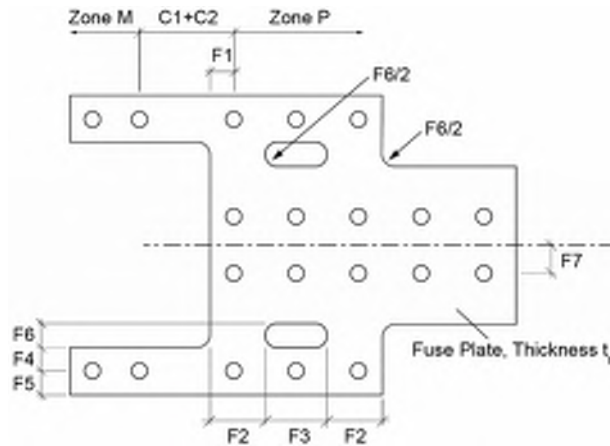


Fig. 7. Fuse plate detail.

2.3 Connection Behavior

The behavior of DuraFuse Frames (DFF) connections has been established through several series of experiments (Reynolds and Uang 2019a; Reynolds and Uang 2019b) that have been published in peer-reviewed journals (Richards 2019; Richards 2021; Richards 2022; Richards and Oh 2019).

Figure 8 shows a connection test with typical behavior [Specimen E1.2 (Reynolds and Uang 2019a)]. The connection remained elastic throughout the 0.00375, 0.005, 0.0075 rad cycles. During the 0.01 rad cycles, slight flaking of the mill scale indicated localized yielding in the yield regions [Fig. 8(a)]. Bolt slip occurred during the 0.015 rad and subsequent cycles. During the 0.02, 0.03, 0.04 rad cycles, inelastic deformations of the fuse plate became more pronounced [Fig. 8(c)-(e)]. The external continuity plates had noticeable curvature at 0.05 rad drift [Fig. 8(f)] but were still primarily elastic (the plates were essentially straight after testing). During the second cycle at 0.06 rad, ductile tearing of the fuse plate initiated [Fig. 8(g)], and during the first excursion to 0.07 rad, the west side of the fuse plate tore through [Fig. 8(h)].

The hysteretic response of DFF connections is similar to other bolted SMF connections, except there is higher post-yield stiffness and no strength degradation at large drifts. Fig. 9 shows the hysteretic response of the connection from Fig. 8. As with other bolted SMF connections, the DFF hysteretic plots have a flatter region in the middle of each large cycle corresponding to bolt slip (Fig. 9). Once bolts returned to bearing, the strength continues to climb. One difference, as compared to other bolted SMF connections is the lack of strength degradation at large drifts. SMF connections that form plastic hinges in the beam have strength degradation after 0.03 or 0.04 rad due to flange and web local buckling of the beam in the plastic hinge region (Uang and Fan 2001). The DFF connections do not experience flange local buckling or local buckling in the fuse plate and maintain flexural strength through large drift cycles until the fuse plate fatigues. Even when the fuse plate tears and the flexural strength of the connection is reduced, the shear strength of the connection remains intact to carry gravity loads. The DFF fuse plates are proportioned so that the ultimate moment at the column face is roughly equal to the nominal moment capacity of the beam (Fig. 9).



Fig. 8. DFF connection behavior [Specimen E1.2 (Reynolds and Uang 2019a)].

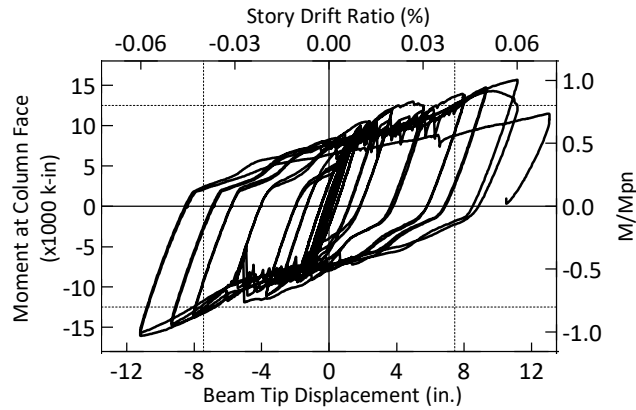


Fig. 9. DFF hysteretic behavior [Specimen E1.2 (Reynolds and Uang 2019a)].

2.4 Design Responsibility

The DuraFuse Frames connection is a proprietary connection used under the supervision of the EOR. Licenses to use the connection are granted on reasonable and nondiscriminatory terms, with the license fee being included in the fabrication bid and paid by the fabricator. When DFF connections are used on a project, the SMF connections are designed by DuraFuse Frames, LLC, based on the member sizes and design loads from the EOR (structural model or other calculations). The connection calculations and connection details/schedules are prepared by DuraFuse Frames, LLC and included in the overall structural design drawings and calculation package assembled by the EOR.

The DFF general specification states: "DuraFuse Frames engineering design responsibilities are limited to design of the beam-to-column connections in the moment frames."

3 Design Methodology

3.1 Overview

The design of DFF SMF consists of selecting beam and column sizes to meet drift and strength requirements (per ASCE 7) while satisfying seismic provision checks (per AISC 341-16 and 358-22), and then proportioning the connection plates, welds, and bolts, to meet strength requirements and ensure that fuse yielding is the governing limit state (per AISC 360-16 and 358-22). The design procedure (AISC 358-22 Section 15.6) addresses these various requirements throughout twenty design steps.

3.2 Drift Checks

Since SMF are inherently flexible, drift requirements often govern beam and column sizes in DFF SMF. When using ASCE 7 to determine the allowable story drift, SMF that are four stories or less, are usually categorized as “structures, other than masonry shear walls structures, four stories or less...with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts”. When SMF are greater than four stories, they are categorized as “all other structures” when determining the allowable story drift.

The equivalent lateral force (ELF) procedure or modal response spectrum analysis (MRSA) is used to determine elastic drifts, as permitted/required in ASCE 7. Elastic drifts are amplified (ASCE 7 12.8-15) to obtain design drifts which can be compared with the allowable. When MRSA is used, special load cases are employed for the drift checks where the natural period used for calculations is not limited to $C_u T_a$ as it is in the ELF procedure and for strength checks (with either ELF or MRSA). Since the drift demands from MRSA are period dependent, iteration is required to ensure that the scaling factors used for the analysis are consistent with the period of vibration of the as-designed frame.

3.3 Strength

3.3.1 Checks Done with ASCE 7 Load Combinations

The equivalent lateral force (ELF) procedure or modal response spectrum analysis (MRSA) is used to determine strength demands on *beams* and *columns*. Unlike the drift checks, the period used in determining strength demands cannot exceed $C_u T_a$. Load effects from lateral forces are combined with gravity load effects using ASCE 7 load combinations. Structural design software (RAM, ETABS, RISA, or similar) is used to compare capacities with demands. LRFD design is used for strength checks.

While the member (beam and column) strength checks are performed using ASCE 7 load combinations, only some of the *connection* strength checks use load combinations. The others are performed using capacity-design principles as described in the next section. Connection strength checks that only use ASCE 7 load combination demands are bolt slip under wind loading and fuse plate rupture under seismic loading. In some other connection strength checks, gravity loads from load combinations are considered in combination with capacity-design seismic forces.

3.3.2 Capacity Design Checks

Most of the connection strength checks involve capacity-design. The connection plates, welds, and bolts are designed to resist the maximum force that can develop in the connection, with limit state capacities calculated per AISC 360. The sub-sections that follow indicate the way that the seismic demands are calculated using capacity design (Sections 3.3.2.1 through Section 3.3.2.3 of this document) and then specify the pertinent checks from AISC 360 that are used to design the connection (Sections 3.3.2.4 of this document).

3.3.2.1 Capacity-design demands at beam bottom flange level

The maximum probable force that will develop at the *bottom flange* level on each side of the connection is computed by assuming the moment at the column face, M_{pr} , is transmitted via a couple at the top and bottom flange levels (Fig. 10). The force V_{fe} is the force transmitted to each side of the connection, so $2^* V_{fe}$ is the magnitude of the force acting at the bottom flange level.

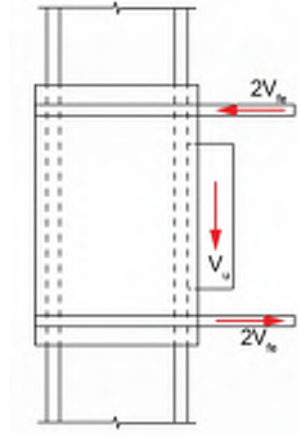


Fig. 10. The bottom flange force associated with M_{pr} .

The bottom flange demand on each side of the connection, V_{fe} , from this capacity-design approach is:

$$V_{fe} = \frac{M_{pr}}{2(d_b + t_p)} \quad (\text{Eq.1})$$

where

M_{pr} = maximum probable moment at the fuse location. This moment shall not exceed M_p and shall not be less than the beam flexural demand defined by the applicable building code. If M_{pr} is less than M_p , analysis shall be performed to demonstrate the connection is fully restrained. Note, the fuse plate is proportioned during the design to have an expected ultimate capacity corresponding to M_{pr} .

d_b = depth of the beam

t_p = thickness of the top plates, fuse plate, and external continuity plates. This thickness is typically equal to the beam flange thickness, t_{bf} , rounded up to the next standard plate size.

M_{pr} is usually equal to M_p , but the definition is flexible to ensure good behavior in situations where the strong-column weak-beam is not required to be satisfied (single-story frames and top-story situations). In situations where the beams are stronger than the column, it is desirable to limit the moment at the joint to less than the beam M_p to preclude column yielding. However, it is still the intent that the connection be fully restrained (FR), so additional calculation is done to justify the FR assumption when M_{pr} is less than M_p .

For joints where two beams frame in, the demands on the external continuity plates at the lower flange level are equal to the sum of the demands from each of the beam ends.

3.3.2.2 Capacity-design demands at beam top flange level

The maximum force that can develop at the *top flange* level will be greater than the bottom flange if drag forces are being passed through the connection (Fig. 11). The drag force on each side of the connection is designated P_d , so $2 \cdot P_d$ corresponds to the total drag force.

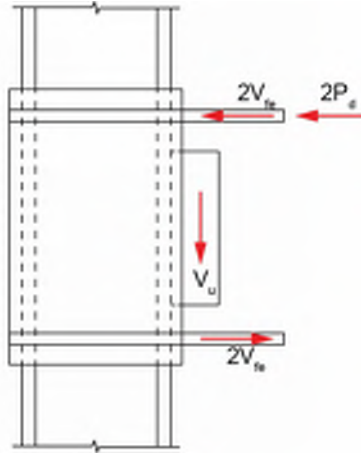


Fig. 11. Demands at top flange level when drag forces are present.

3.3.2.3 Capacity-design demand at shear tab

The required shear for the tab, V_u , is computed in accordance with 2.5 of AISC 358-22 which states “the required shear strength of the beam shall be taken as the shear at the plastic hinge, V_h , plus shear due to gravity load applied to the beam between the plastic hinge and the face of the column, using the load combination in Section 2.4.4.”

3.3.2.4 Capacity-design strength limit state checks

The capacity-design demands determined in the previous sections are used to check connection limit states per AISC 360-22. These various limit state checks are described in the design steps in AISC 358-22 Chapter 15 Section 15.6 (See Appendix for example calculations).

3.3.2.5 Expected fuse yield strength

In the calculations, the yield force of the fuse based on the final fuse plate geometry is documented in Step 19 and demonstrated to be less than V_{fe} used in the design of the other elements. The ratio of V_{fe}/V_y is usually around 1.9. This reflects material overstrength, material strain hardening, and the indirect effect of tensioned web bolts that increase moment capacity at the connection. The procedure for proportioning fuses (see AISC 358-22 Section 15.6 Step 19 and associated Commentary) has calibrated parameters based on the experimental testing.

3.4 Seismic Provisions

In addition to drift and strength checks per ASCE 7 and AISC 360, the seismic provisions give additional checks to ensure ductile behavior. The checks that pertain to the DFF elements are the strong-column weak-beam check, lateral bracing checks, and member width-thickness checks.

3.4.1 Strong-Column Weak-Beam

The column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*. The value of ΣM^*_{pb} shall be taken equal to $\Sigma(M_{pr} + M_{uv})$ where M_{pr} is the maximum probable moment at the fuse, and M_{uv} can be computed as $V_b(d_c/2)$, where d_c is the depth of the column and V_b can be computed as $2M_{pr}/L_h + V_g$, where L_h is the clear distance between column faces and V_g is the beam shear caused by gravity loads based on the load combination $1.2D + f_1L + 0.2S$, where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5. The value of ΣM^*_{pc} shall be the sum of the projections of the nominal flexural strengths (M_{pc}) of the column above and below the joint, at the potential hinge location located one quarter of the column depth above and below the cover plates edges.

3.4.2 Lateral Bracing

3.4.2.1 Beam Stability Bracing

There are no requirements for stability bracing of beams or joints beyond those in AISC 360-22.

The prequalification experiments for the DuraFuse Frames connection have been conducted without lateral bracing near the beam-column connection or along the length of the beam. The only beam restraint in the tests was at the beam tip (actuator location). The DuraFuse Frames connection is much less susceptible to beam lateral-torsional buckling or column twisting because the beam does not yield and there is no local buckling in the fuse plate to disrupt symmetry of the load path at the connection. Thus, the beam stability bracing can be determined using the AISC *Specification*, neglecting the prescriptive requirements for SMF that assume beam yielding.

Table 1 summarizes the unbraced lengths of test specimens, relative to the unbraced length parameters in AISC 341-16 and AISC 360-16. The unbraced lengths in the test specimens, L_b , far exceeded the bracing in AISC 341-16 for highly ductile beams [$0.95r_yE/(R_yF_y)$], and the plastic unbraced length, L_p [$1.76r_y\sqrt{E/F_y}$].

Table 1. Unbraced lengths of test specimens.

Series	Beam Size	Unbraced Length, L_b ^a (ft)	$0.95r_yE/(R_yF_y)$ ^b (ft)	L_p (ft)
E	30×99	15.5	8.8	7.42
F	40×167	15.5	10	8.48
G	21×50	15.5	5.4	4.59
H	36×232	20.5	10.9	9.25

a. Distance from the column centerline to the beam tip in the experiments

b. AISC 341 bracing requirement for highly ductile beams that are expected to yield

In DFF design checks, beam stability bracing is provided based on the maximum force that can develop in the beam (capacity-design) and the stability requirements in AISC 360-16. DFF calculations include a maximum unbraced length of the beam based on capacity design.

3.4.2.2 Column Stability Bracing

Lateral bracing of the columns shall comply with AISC 341-16. The fuse plates at the connection are proportioned to preclude column yielding, so only restraint at the top beam flange level is required. The cover plates in the column create a torsionally rigid segment of the column that provides indirect lateral restraint to the column in the regions immediately outside the torsionally rigid segment.

3.4.3 Member Width-Thickness Requirements

For beams, there are no limits on the web width-to-thickness ratio beyond those listed in the AISC *Specification*, since the connection design precludes beam yielding. The beam flange width-to-thickness ratio shall not exceed λ_p per AISC *Specification* Table B4.1b.

For columns, width-to-thickness ratios for the flanges and web shall conform to the requirements for highly ductile sections in the AISC *Seismic Provisions*.

3.5 Prequalification Limits

3.5.1 Beams

Beams shall satisfy the following limitations:

(1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3 in AISC 358-22.

(2) Beam depth shall be limited to a maximum of W40 (W1020) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.

(3) For HCAI, the beam weight shall be limited to a maximum of 232 lb/ft, and up to a maximum of 397lb/ft with a welded top plate detail, since CBC 2205.A.4.9 does not allow extrapolation above the heaviest beams tested.

The limitations on beam sizes are based on the extent of experimental testing that was considered by CPRP for AISC 358-27. The DuraFuse Frames connection has been investigated with a variety of beam sizes. The smallest beam size that has been tested with fuse plates with extensions, similar to Fig. 15-1, is W21×50. On the bigger side, more than ten tests have been performed with W33, W36, and W40 beams. The deepest beam that had been tested at the time of AISC 358-22 review was W40×167 (Reynolds and Uang 2019a). The heaviest beam that had been tested at the time of AISC 358-22 review was W36×232 (Reynolds and Uang 2019b). In August of 2021 the heaviest beam that has been tested and reviewed by AISC 358 CPRP committee is the W40x397 (Lee and Uang 2021).

Although the AISC *Seismic Provisions* permit limited increase in beam depth compared to the maximum sections tested, the prequalification limit for maximum beam depth was not extrapolated beyond the test data for W40×397 to be consistent with other connections in AISC 358. The strain demands on the fuse plate are directly related to the beam depth, so it is conservative to base the beam depth prequalification limit on the maximum tested specimen.

The heaviest beams tested at the time of AISC review were two K-series beams that were W40x397. Both beams met the HCAI acceptance criteria by completing at least two cycles at 0.04 rad drift and 0.03 rad inelastic rotation (Lee and Uang 2021). The tests indicated ultimate rotational capacity beyond 0.06 rad. The K-series tests were supported by others in the H & F-series tests that indicated ultimate rotation capacity beyond 0.06 rad (Reynolds and Uang 2019a and 2019b).

3.5.2 Columns

Columns shall satisfy the following limitations: 01/15/2025

(1) Columns shall be any of the shapes permitted in AISC 358 Section 2.3.

(2) Rolled shape column depth shall be limited to W40x (W1020) . Column depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes. Built-up box-columns shall not have a width exceeding 30 in (762 mm).

The DuraFuse Frames connection has been investigated with a variety of column shapes and sizes. The W14 columns that have been tested are: W14×38 and W14×68. The W21 column that has been tested is W21×132. The W24 columns that have been tested are: W24×103 and W24×250. The W36 (W920) columns that have been tested are: W36×150 and W36×231 (Reynolds and Uang 2019a). The box columns that were tested were 40 in. deep, 30 in. wide, with a plate thickness of 2 in (Lee and Uang 2021). The connection details for box columns and HSS columns are the same, so testing of box columns also represents HSS columns. The prequalification limit for column depth for DuraFuse Frames connections is based on the maximum tested specimen.

3.5.3 Plates

Plates shall satisfy the following limitations:

(1) All connection plates shall be fabricated from structural steel that complies with ASTM A572/A572M Grade 50 (Grade 345). Fuse plates shall be fabricated from plates that have mill certified tensile strengths less than or equal to 85 ksi, unless independent material testing determines that the tensile strength is less than or equal to 85 ksi.

- (2) The thickness of the external continuity plates, top plates, and fuse plates shall not exceed 2 in. (50 mm) and shall not be less than 0.5 in. (13 mm).
- (3) The width of the yielding regions in the fuse plates shall not exceed 4.0 in. (102 mm) and shall not be less than 1.5 in. (38 mm)
- (4) The width-thickness ratio of the yielding regions in the fuse plates shall not exceed 4.25 and shall not be less than 1.5.
- (5) The width-depth ratio of the yielding regions in the fuse plates shall not exceed 1.25 and shall not be less than 0.5.
- (6) The roughness of all thermal cut surfaces on the plates (including fuse plates) shall be no greater than an ANSI surface roughness of 1000 micro-inches. Roughness exceeding this value or gouges not more than 3/16 in. shall be removed by machining or grinding. The fuse plates used for prequalification tests were fabricated with thermal cutting.

All of the tests that have been performed for the DuraFuse Frames connection have used plate material that satisfies ASTM A572/A572M Gr. 50 for the cover plates, fuse plates, top plates, and external continuity plates (Reynolds and Uang 2019a; Reynolds and Uang 2019b). In the absence of experimental data for other plates, prequalification is limited to plates that satisfy ASTM A572/A572M Gr. 50.

The fuse plates have the additional requirement that the material not have a tensile strength greater than 85 ksi. Since there is no upper limit on the tensile strength of ASTM A572/A572M Gr. 50 steel in the ASTM specifications, it is prudent to have some protection against steel that is too strong in the fuse. The strongest steel that was used for fuses in testing had a mill certified tensile strength of 86.5 ksi (Reynolds and Uang 2019a).

A variety of geometries for the yielding regions (YR) of the fuse plates have been investigated and found to provide sufficient deformation capacity. The limits on the thickness and width of the YR are based on the dimensions of the YR that have been tested.

3.5.4 Bolts

Bolts shall satisfy the following limitations:

- (1) Bolts shall be arranged symmetrically about the axis of the beam and shall be limited to two lines of bolts in each beam flange.
- (2) Standard holes shall be used in the beam flanges and in the external continuity plates. Holes in the top plates and fuse plate shall be oversized except for the short-slotted holes indicated in Fig. 6
- (3) Bolt holes in beam flanges, fuse plates, and other connection plates shall be made by drilling, sub-punching and reaming, laser cutting, plasma cutting, or waterjet cutting.
- (4) All bolts shall be installed as pretensioned high-strength bolts.
- (5) Bolts shall be ASTM F3125 Grade A325, Grade A325M, Grade A490, Grade A490M, Grade F1852, or Grade F2280 assemblies. Threads shall be excluded from the shear plane. Bolt diameter is limited to 1-1/4 in (32 mm) maximum.
- (6) Faying surfaces shall have a Class A slip coefficient but shall not have a surface roughness that achieves a friction coefficient in excess of 0.5 (Class B)

3.6 Drags and Collectors

3.6.1 In-line

For in-line drags and collectors, the drag force is designed to be transmitted at the joint at the top flange level. The drag/collector force is combined with the maximum force at the top flange that can be developed by the fuse plate (Fig. 11). This combined force is used when checking bolts, welds, and connection plates (see Section 2.3.2 of this report). The drag force does not reduce the moment capacity of the connection, since the moment capacity is governed by the fuse strength at the bottom flange level and drag forces are not transferred at the bottom flange level.

Beams that are used as drags and collectors should be checked by the EOR for combined axial, moment, and shear. This is independent of the connection design performed for the DFF connections.

3.6.2 Orthogonal

For orthogonal drags and collectors, the drag force is designed to be transmitted through the joint at the top flange level. The geometry of the external continuity plate is adjusted to simplify the load path of the orthogonal forces through the connection. Fig. 12 illustrates the detail (shown for cantilever beam, but the same detail applies for orthogonal collectors). The thickness of the external continuity plate and the weld sizes are checked based on the demands from the moment frame ($V_{fe} + P_d$) in combination with the orthogonal demands, $P_{d,o}$.

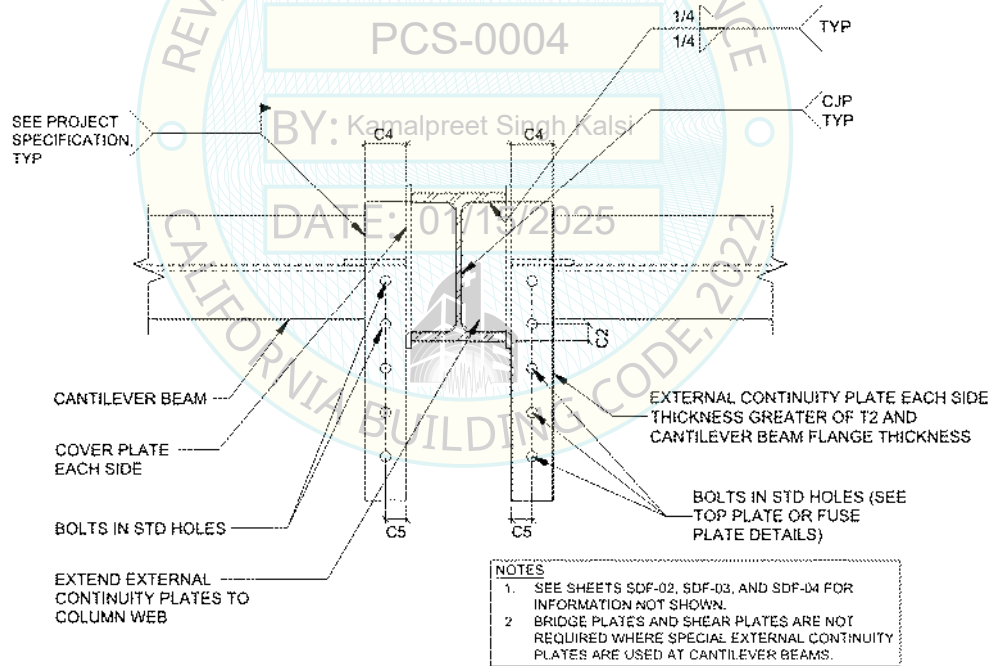


Fig. 12. Special external continuity plate for orthogonal drags or cantilevers.

The equations used for the design checks are derived below. Figure 13 summarizes terms that are used in the derivations.

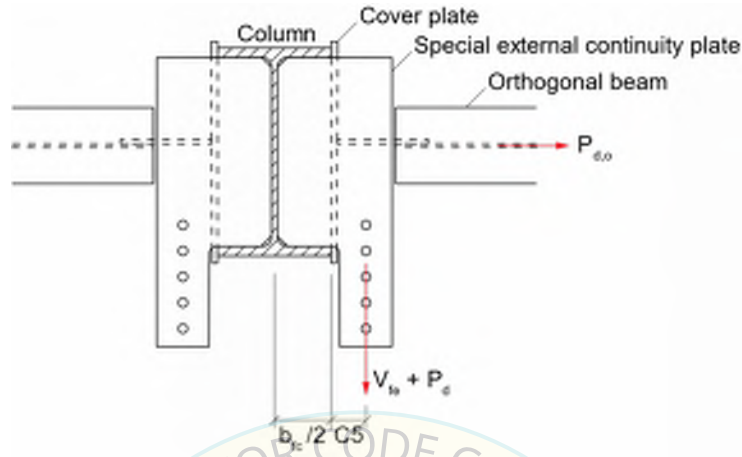


Fig. 13. Terms used in orthogonal load equations.

The demands on the special external continuity plate are summarized in Fig. 14. The derivations conservatively neglect any force transferred by the fillet welds between the special external continuity plate and the column flanges.

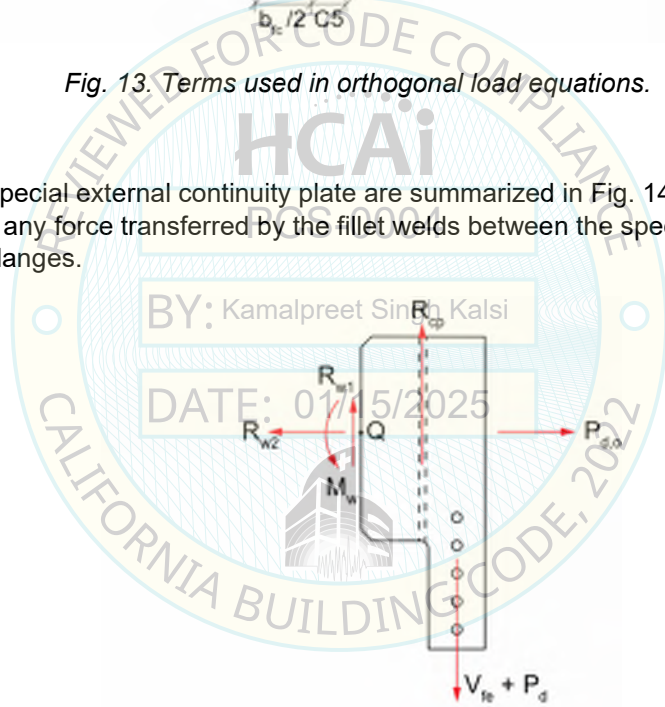


Fig. 14. Demands on special external continuity plate.

The demands in Fig. 14 are related through three equilibrium equations and one compatibility equation.

$$\Sigma F_{x,dir} = 0, \quad R_{w2} = P_{d,o} \quad (\text{Eq. 2})$$

$$\Sigma F_{y,dir} = 0, \quad R_{cp} + R_{w1} = V_{fe} + P_d \quad (\text{Eq. 3})$$

$$\Sigma M_{@Q} = 0, \quad (V_{fe} + P_d) \left(\frac{b_{fc}}{2} + C5 \right) - R_{cp} \left(\frac{b_{fc}}{2} \right) = M_w \quad (\text{Eq. 4})$$

$$\frac{R_{cp}}{t_p} = \frac{R_{w1}}{0.5t_{wc}} \quad (\text{Eq. 5})$$

The compatibility equation (Eq. 5) results in similar shear stress/deformation in the column web and the cover plates, and reflects the observation of essentially uniform panel zone deformation.

Equations 2-5 are used to solve for the four unknowns: R_{cp} , R_{w1} , R_{w2} , and M_w .

Combining Eq. 3 and Eq. 5:

$$R_{cp} = \frac{(V_{fe} + P_d)}{\left(1 + \frac{0.5t_{wc}}{t_{cp}}\right)} \quad (\text{Eq. 6})$$

Then from Eq. 3:

$$R_{w1} = (V_{fe} + P_d) - R_{cp} \quad (\text{Eq. 7})$$

Equations 2 and 4 provide direct calculations for R_{w2} and M_w , respectively, as is.

The capacity of the special external continuity plate is checked at two sections, at the cover plate and at the beam web. At both locations, the connections are CJP welded, so member limit states govern.

A free-body diagram with a cut at the cover plate section is shown in Fig. 15.

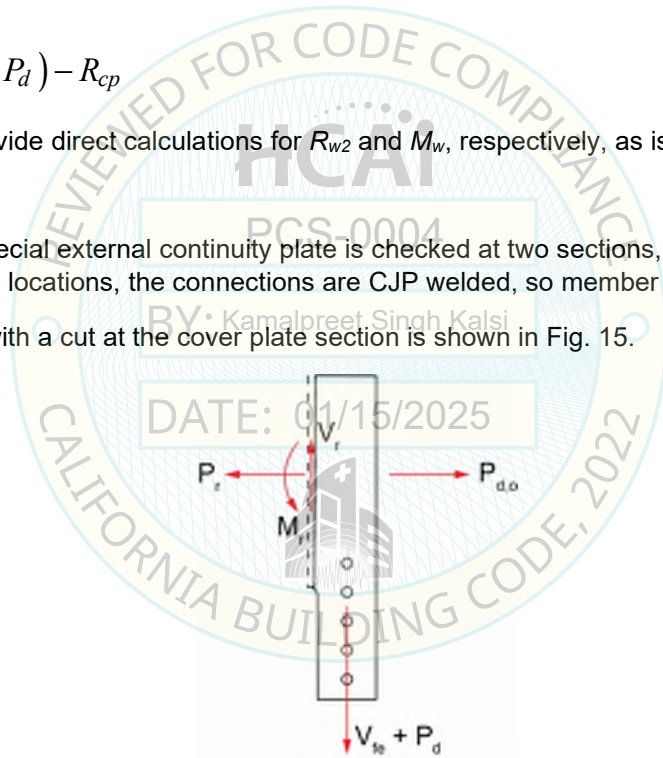


Fig. 15. Special external continuity plate with a cut at the cover plate section.

The flexural, normal, and shear demands on the section (Fig. 15) are expressed in terms of previously defined forces:

$$M_r = (\Sigma V_{fe} + P_d)[C5] \quad (\text{Eq. 8})$$

$$P_r = P_{d,o} \quad (\text{Eq. 9})$$

$$V_r = (\Sigma V_{fe} + P_d) \quad (\text{Eq. 10})$$

The capacities of the plate, M_c , P_c , and V_c are determined per AISC 360.

The plate is checked per the general connection interaction equation in the AISC Manual (Eq 9-1 in the fifteenth edition).

$$\frac{M_r}{M_c} + \left(\frac{P_r}{P_c}\right)^2 + \left(\frac{V_r}{V_c}\right)^4 \leq 1.0 \quad (\text{Eq. 11})$$

A similar check is performed at the section at the column web section (Fig. 16).

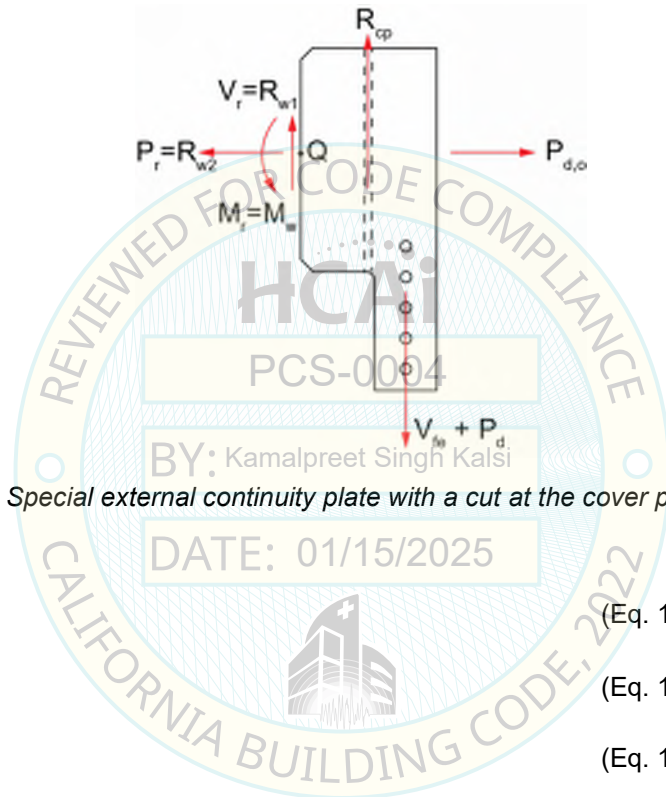


Fig. 16. Special external continuity plate with a cut at the cover plate section.

$$M_r = M_w \quad (\text{Eq. 12})$$

$$P_r = R_{w2} \quad (\text{Eq. 13})$$

$$V_r = R_{w1} \quad (\text{Eq. 14})$$

$$\frac{M_r}{M_c} + \left(\frac{P_r}{P_c}\right)^2 + \left(\frac{V_r}{V_c}\right)^4 \leq 1.0 \quad (\text{Eq. 15})$$

3.7 Orthogonal Gravity Cantilevers

For orthogonal gravity cantilevers, the geometry of the external continuity plates is adjusted to accommodate cantilevers. Figures 17-19 illustrate the details for cases where the orthogonal cantilever beam is the same depth as the DFF beam, is shallower than the DFF beam, or is deeper than the DFF beam.

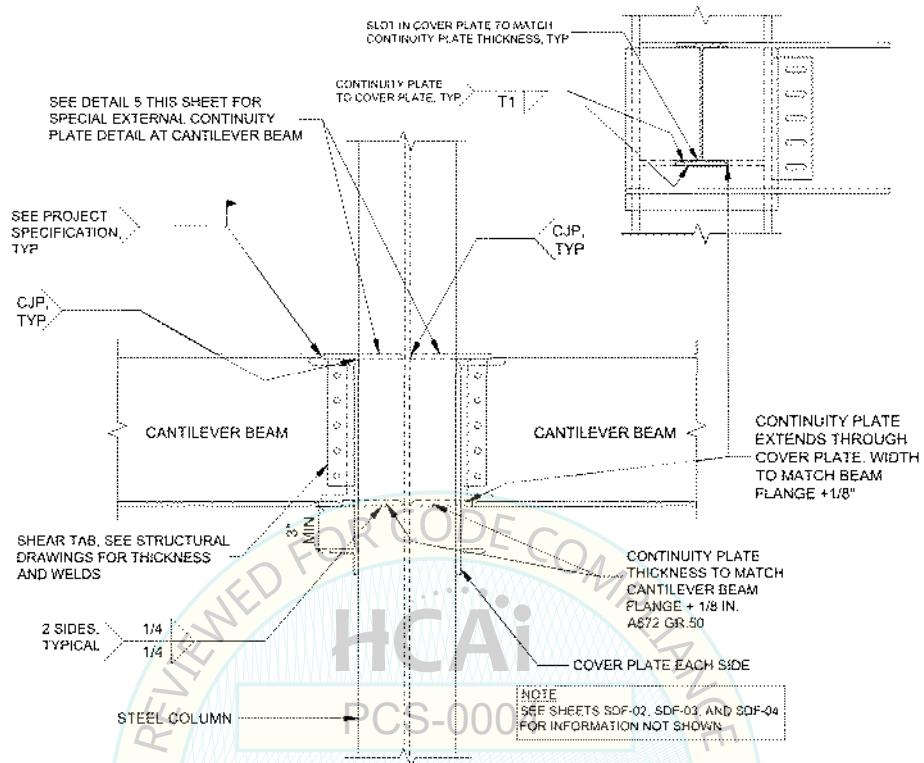


Fig. 17. Orthogonal gravity cantilever when cantilever depth is less than DFF beam depth.

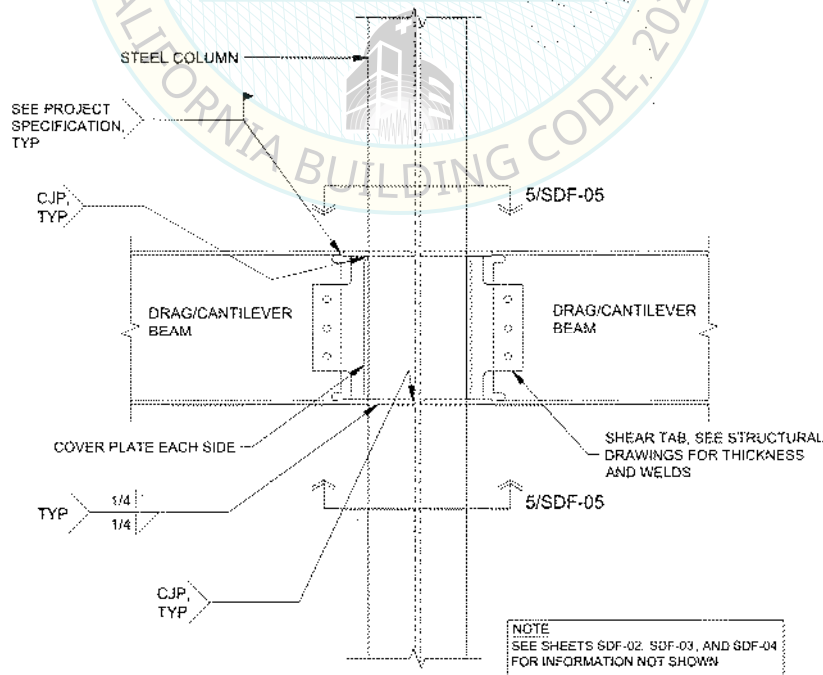


Fig. 18. Orthogonal gravity cantilever when cantilever depth equal to DFF beam depth.

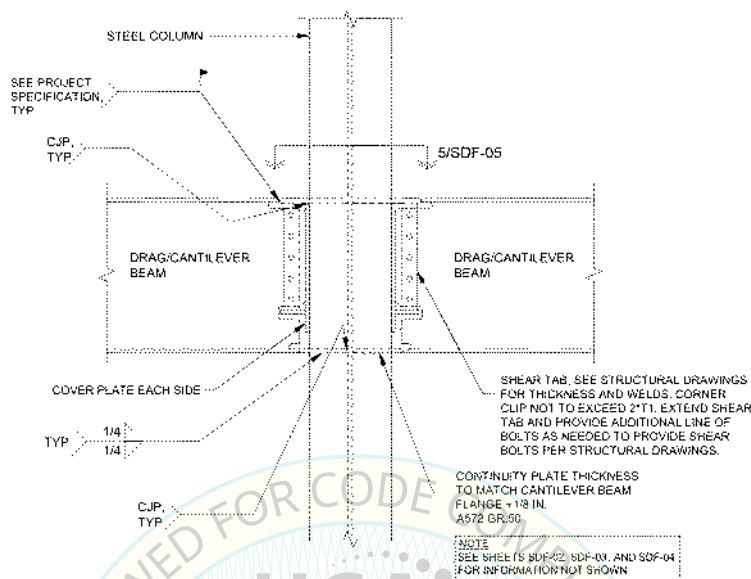


Fig. 19. Orthogonal gravity cantilever when cantilever depth is greater than DFF beam depth.

In all cases (Figs. 17-19), the top flange of the cantilever beam is directly attached to the external continuity plates of the DFF connection. The calculations to check the external continuity plate are the same as for orthogonal drags, but the force $P_{d,o}$ is taken as M_u/d_b where M_u is the design force for the cantilever beam.

In all cases (Figs. 17-19), a continuity plate is provided in the column at the bottom flange level to help transmit compression loads and mitigate cover plate deformations.

3.8 In-Plane Gravity Cantilevers

When a non-seismic gravity cantilever beam is present on the opposite face of a column with a one-sided special moment frame connection, the EOR may choose to use a two-sided DFF connection. In these situations, the bottom plate for the cantilever connection does not have the cut-outs defined by F3 (Fig. 7). The connection calculations combine the gravity load effects from the cantilever side with the capacity-design effects for the special moment frame connection on the other side.

3.9 Sloped and Stepped Connections

For roof connections that are sloped to accommodate drainage (up to a 1:12 slope), the column detail of Fig. 3 is modified to that shown in Fig. 20. The slope does not impact the calculation of the maximum forces being delivered at the top and bottom flange level (Equations in 2.3.2 are still valid). The slope does increase the distance from the centroid of the shear tab bolt group to the face of the column. The adjusted distance is used when computing the moment on the shear tab weld. The slope increases the length of W2 at the top external continuity plate and the length of W3 on the shear tab, but those increased weld lengths are conservatively neglected in calculations.

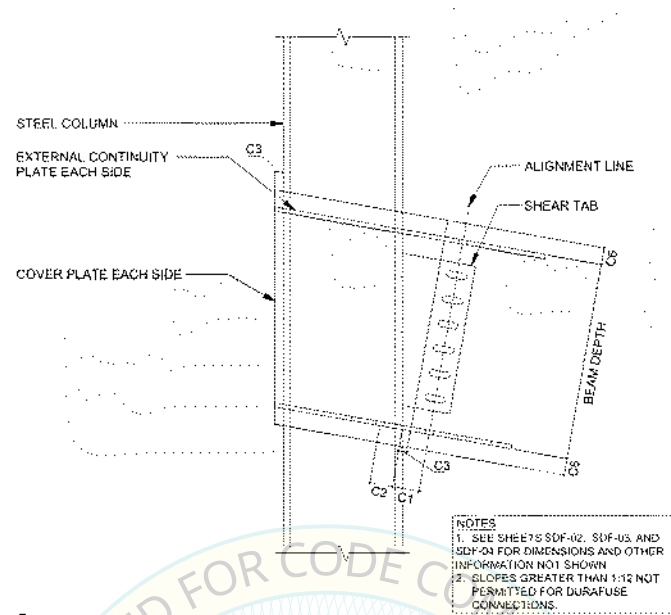


Fig. 20. Connection plates when the beam is sloped.

If DFF beams of different depths frame into the same joint, the detail shown in Fig. 21 is used. In this configuration, the length of W1 increases (cover plate to column weld), but the demands on W1 and the cover plate are no higher than when the external continuity plates are aligned. Similarly for cases with dropped beams (Fig. 22).

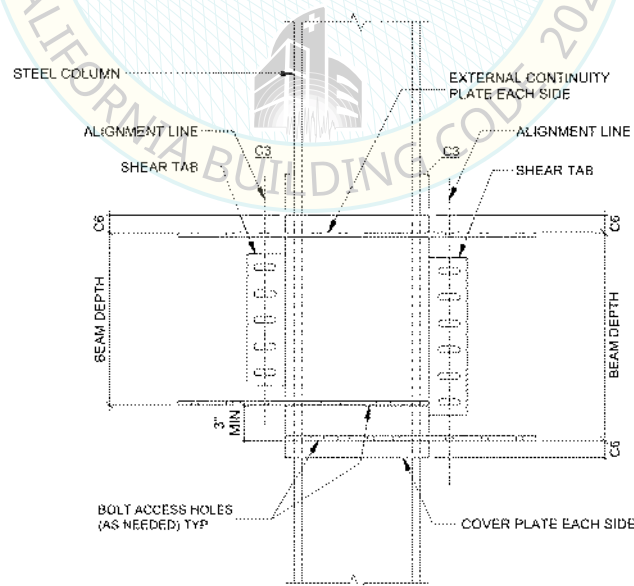


Fig. 21. Column detail for DFF beams of different depth.

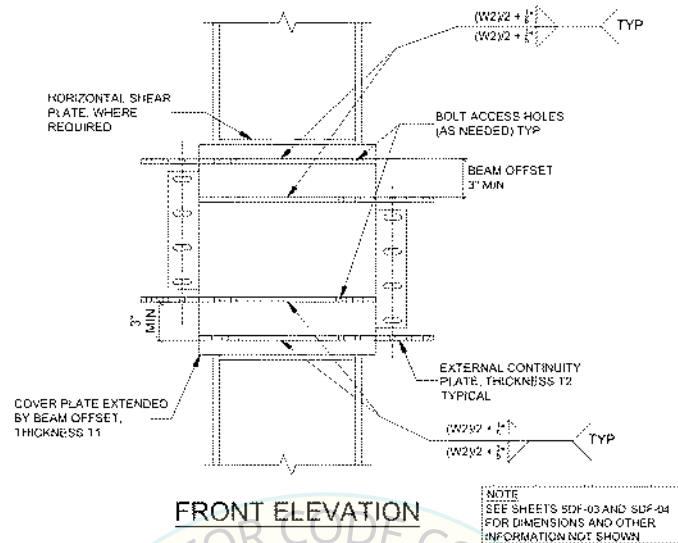


Fig. 22. Column detail for DFF beam that is dropped.

3.10 Example Calculations Summary Sheet

In DFF calculation submittals, each connection ID will have a summary sheet that lists the capacity and demand for all the limit states checked in each step of the design procedure (AISC 358-22 Section 15.6). Table 2 is an example of the summary table. In addition, detailed calculations for each limit state for each connection can be provided.

The calculations summary sheet and detailed calculations are organized by 20 design steps that apply to all connections (AISC 358-22 Section 15.6) followed by miscellaneous calculations that apply in some circumstances. Miscellaneous calculations include: bridge plate design checks (when bridge plates are present), shear plate design checks (when shear plates are present), FR assumption check (if M_{pr} is less than M_p , see 3.3.2.1 of this guide), rigid panel zone assumption check (if rigid panel zones have been assumed in the building model, see 4.4 of this guide), and external continuity plate checks for out-of-plane forces (if there are orthogonal drag loads or orthogonal cantilevers, see 3.6.2 and 3.7 of this guide).

Table 2. Calculation summary table for DFF connection.

Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 7.2$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 5.6$	OK
	Column-beam moment ratio	$\Sigma M_{pc}^* = 10142$	$\Sigma M_{pc}^* = 5258$	OK
2	Maximum probable moment	$M_u = 2257$ k-in	$M_{pr} = 4749$ k-in	0.48
3	Cover plate shear yielding	$R_u = 112$ kips	$\phi R_n = 348$ kips	0.32
	Cover plate thickness	$t_{cp} = 0.5$ in	$t_{req} = 0.4$ in	OK
4	Beam net section check	$M_{pe} = 5247$ k-in	$M_r = 5437$ k-in	OK
5	Beam flange bolt shear failure	$R_u = 112$ kips	$\phi R_n = 189.4$ kips	0.59
	Bolt slip for wind load	$R_u = 41.6$ kips	$\phi R_n = 70.6$ kips	0.59
6	Alignment line location, C1	$C1_{req} = 6.4$ in	$C1 = 2.4$ in	OK
7	Weld 1 strength	$r_u = 5.5$ kip/in	$\phi r_n = 10.1$ kip/in	0.55
8	Weld 2 strength	$r_u = 7.9$ kip/in	$\phi r_n = 8.4$ kip/in	0.94
9	External continuity plate rupture: Mode 1	$P_u = 67.2$ kips	$\phi P_n = 70.1$ kips	0.96
	External continuity plate rupture: Mode 2	$P_u = 89.6$ kips	$\phi P_n = 99.3$ kips	0.90
	External continuity plate rupture: Mode 3	$P_u = 112$ kips	$\phi P_n = 166.3$ kips	0.67
	External continuity plate rupture: Mode 4	$R_u = 89.6$ kips	$\phi R_n = 108$ kips	0.83
	External continuity plate rupture: Mode 5	$R_u = 112$ kips	$\phi R_n = 139$ kips	0.81
10	Beam shear	$V_u = 48$ kips	$\phi V_n = 217.4$ kips	0.22
11	Beam block shear	$R_u = 224$ kips	$\phi R_n = 229.8$ kips	0.97
12	Web bolt shear	$R_u = 48$ kips	$\phi R_n = 151.5$ kips	0.32
13	Shear tab rupture through net section	$f_u = 14.4$ ksi	$\phi F_u = 48.8$ ksi	0.29
	Shear tab weld failure	$r_u = 14.4$ kips	$\phi r_n = 16.7$ kips	0.86
15	Top plate for shear yielding	$R_u = 112$ kips	$\phi R_n = 225$ kips	0.50
	Top plate for shear rupture	$R_u = 112$ kips	$\phi R_n = 137.1$ kips	0.82
16	Top plate for tensile rupture in extensions	$P_u = 44.8$ kips	$\phi P_n = 45.7$ kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -0.2$ in	$P2 = 2.1$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 3.8$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 89.6$ kips	$\phi P_n = 153$ kips	0.59
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 89.6$ kips	$\phi P_n = 133.1$ kips	0.67
19	Fuse yielding region dimension	$F2_{req} = 2.2$ in	$F2 = 2.1$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 0.9$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 89.6$ kips	$\phi R_n = 135$ kips	0.66
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 89.6$ kips	$\phi R_n = 109.7$ kips	0.82
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	$K_{req} = 1222350$ k/in	$K_s = 3042440.7$ k/in	OK
	Rigid panel zone assumption	$K_{req} = 4074499.9$ k/in	$\beta_{PZ} = 10908237.2$ k/in	OK
External continuity plates with out-of-plane forces	$Inter_{limit} = 1.00$	$Inter = 0.48$	OK	
Controlling Demand-Capacity Ratio				0.98

4 Computer Modeling

4.1 Overview

Modeling guidance for DuraFuse Frames for specific software is provided through a series of technical bulletins (www.durafuseframes.com). The general methods that apply, regardless of the software, are summarized in this section.

4.2 Beam and Column Elements

For special moment frames with relatively deep columns and beams, the shear deformations in the columns and beams are significant, relative to the flexural deformations, and should be considered in the model through the use of Timoshenko beam elements (or similar).

When modeling DFF beams, it is conservative to use the same cross-sectional properties of the beams and columns along their entire length, even though some stiffening occurs near the connections due to connection plates. When justified by connection analysis a non-prismatic beam section can be modeled to include the additional stiffening conditions.

4.3 Beam-to-Column Connections

The DuraFuse Frames beam-to-column connection is modeled as a fully restrained (FR) connection. This classification is based on the experiments used for prequalification, from which the connection stiffness was derived (Richards 2022). When building the computer model, the beams are considered fixed to the columns.

4.4 Panel Zones

The panel zones in DFF are stiffened by cover plates (Fig. 3). Experiments have demonstrated that, with deeper columns, the stiffened panel zones are essentially rigid relative to the beam and column (Reynolds and Uang 2019a).

DuraFuse Frames panel zone may be modeled as rigid, when the computed panel zone stiffness, converted to a rotational spring, is at least ten times larger than the flexural stiffness of the beam (ASCE 2017). Otherwise, the flexibility of the panel zone shall be represented by rigid end zones for the beam and column elements and a scissor spring (Charney and Marshall 2006).

5 Fabrication and Erection

5.1 Overview

The DuraFuse Frames connections are fabricated by the same fabricator that is providing the structural steel package for the building. DuraFuse Frames grants a project-specific license to the fabricator to use the proprietary DFF connection (upon payment of the DFF license fee by the fabricator).

The DuraFuse Frames connections are erected by the same erector that is providing structural steel erection services for the building.

5.2 Fabrication and Erection

All welders, tack welders, and welding operators shall be qualified in conformance with AWS D1.1, Clause 4, Part C. Welders and welding operators performing welds as described in AWS D1.8 Clause 5.1.1 shall be subjected to Supplemental Welder Qualification Testing in accordance with AWS D1.8 Chapter 5.

Welding shall be performed under a Welding Procedure Specification (WPS) in accordance with AWS D1.1, which shall be prepared for every different welding application including welding position, welding process, electrode manufacturer, filler metal trade name for the electrode type selected, and other essential variables as defined in AWS D1.1. None of the welds in the DuraFuse Frames connection are classified as demand critical.

Erection of DuraFuse Frames shall be in accordance with Chapter M of AISC 360, AISC 303, Chapter I of AISC 341, and Chapter 17A and 22A California Building Code.

5.3 QA/QC

A quality assurance plan conforming to 2022 CBC Section 1704A.2, AISC 360 Chapter N, AISC 341 Chapter J, and AWS D1.8 Clause 7 shall be included in the construction documents by the registered design professional and approved by the building official.

Where lamellar tearing in base metal is observed by visual inspection, the steel fabricator shall repair per AWS D1.1 Section 5.14.

For shear plate welds that encroach on the K-area, visual inspection is required no sooner than 48 hours after welding per AWS D1.8, Section 7.4.

5.4 Protected Zones

The protected zone of the connection consists of only the yielding regions on the fuse plates (as indicated in the drawings). None of the other elements (beam, column, connection plates) experience significant inelastic deformations up to 0.04 rad drift or 0.03 rad inelastic rotation of the connection (Reynolds and Uang 2019a; Reynolds and Uang 2019b).

5.5 Fuse Material Check

As indicated in Section 3.5.3 of this guide, the fuse plates have the additional requirement that the material not have an MTR tensile strength greater than 85 ksi, unless independent testing confirms that the tensile strength is lower. The steel fabricator is responsible to report the MTR strength for the fuses or report independent testing to demonstrate compliance.

The surface roughness of the fuse plates shall be no greater than an ANSI surface roughness of 1000 micro-inches. Roughness exceeding this value or gouges not more than 3/16 in. shall be removed by machining or grinding. The curved cuts on the fuse plate shall be defined by a smooth radius (no faceted curves).

5.6 Welding

All welding shall be performed using E70 electrodes. The weld filler metal and associated welding process for all fillet welds shall meet the requirements of any of the following:

- E70T-6, E71T-8, or E70TG-K2 for flux-cored arc welding (FCAW)
- E7XT-9 for flux core arc welding (FCAW) with gas shielding
- F7A2-EXXX for submerged arc welding (SAW)
- E7018 stick electrode for shielded metal arc welding (SMAW)
- ER70S-X, E70C-XM, or E70C-XC for gas metal arc welding, except for short circuit transfer.

All weld filler material shall satisfy the requirements specified in Clause 6.3 of the Structural Welding Code-Seismic Supplement (AWS D1.8/D1.8M) including the minimum Charpy V-notch (CV) toughness 20 ft-lb (27 J) at a temperature lower than 0°F (-18°C) as indicated in Section A3.4 of AISC 341-16, and 40 ft-lb impact strength at 70°F as determined by AWS classification methods of manufacturer certification. All weld filler metal shall satisfy a maximum diffusible hydrogen content requirement per AWS D1.1 and D1.8 Section 6.3 of 16 milliliters of hydrogen per 100 grams of weld metal (H16) or less.

5.7 Bolting

Bolts shall be pretensioned high-strength bolts conforming to ASTM F3125 Grade A490 or A490M, or Grade A325 or A325M, or Grade F2280 or F2280M, or Grade F1852 or F1852M. Bolt diameter is limited to a maximum of 1¼ inches (32 mm).

Shims with a maximum overall thickness of ¼ inch (6.4 mm) may be used between the top plates and bars or beams and between the fuse plate and bars or beams. Shims, if required, may be finger shims or may be produced with drilled or punched holes. Experiments have demonstrated the same performance for connections with and without shim plates (Reynolds and Uang 2019a).

5.8 Shape and Fabrication Tolerances

To account for shape tolerances (actual width more than nominal), column flange trimming via grinding is permitted. The roughness of ground surfaces shall be no greater than an ANSI surface roughness of 1000 micro-inches. Transitions shall be 10:1 or greater. The trimming shall not reduce the flange width to less than the nominal design dimension.

To account for shape tolerances (actual width less than nominal but within AISC tolerances) shims are permitted between the cover plates and the column flanges. The shim material shall be the same grade as the cover plate. Fillet weld size shall be increased by the size of the shim.

A gap of up to and including ¼ in. (6 mm) between the top plate and the external continuity plate or beam flange and between the fuse plate and the external continuity plate or beam flange may be closed through deformation of the plates.

5.9 Specification Subsection and TIO

5.9.1 Specification Subsection

DuraFuse Frames fall under the requirements of the Structural Steel Framing specification for a project and does not require its own specification subsection. SEOR shall include the following DuraFuse specific subsection within the Structural Steel Framing specification.

1.X Requirements for use of Patented Moment Connection

- A. Project includes use of DuraFuse Frames® moment frame connections.
- B. Contractor shall be responsible for payment of license fee to DuraFuse Frames.

C. DuraFuse Frames® have requirements beyond this Section affecting submittals, materials, tolerances, details of fabrication and erection, quality control and quality assurance. Conform to these additional requirements designated on Drawings.

1. Where designated requirements are less stringent than the requirements of this Section, the more stringent requirements of this Section shall apply except as otherwise approved by Owner’s Representative.
2. Additional requirements are only applicable to DuraFuse Frames® connections and connected members.

5.9.2 TIO

On-site special inspection for DuraFuse Frames shall include:

C-S7: Fuse plate curved cuts shall be defined by a smooth radius (no faceted curves).

C-S8: Fuse plates shall have a surface roughness no greater than ANSI surface roughness of 1000 micro-inches.

Index #	Stage 1 Required (Select with "X")	ON-SITE SPECIAL INSPECTIONS	Samples of Test & Inspection Reports Included	Select with required OPA
C-85		CBC 1705A.2, 1705A.12.2, & 1705A.13.3 Cold-formed steel light frame construction		
C-86		Steel CBC 1705A.13 Special inspections for seismic resistance		
C-87	X	Steel DuraFuse Frames Moment Connection Fuse plates curved cuts shall be defined by a smooth radius (no faceted curves).		
C-88	X	Steel DuraFuse Frames Moment Connection Fuse plates shall have a surface roughness no greater than ANSI surface roughness of 1000 micro-inches.		

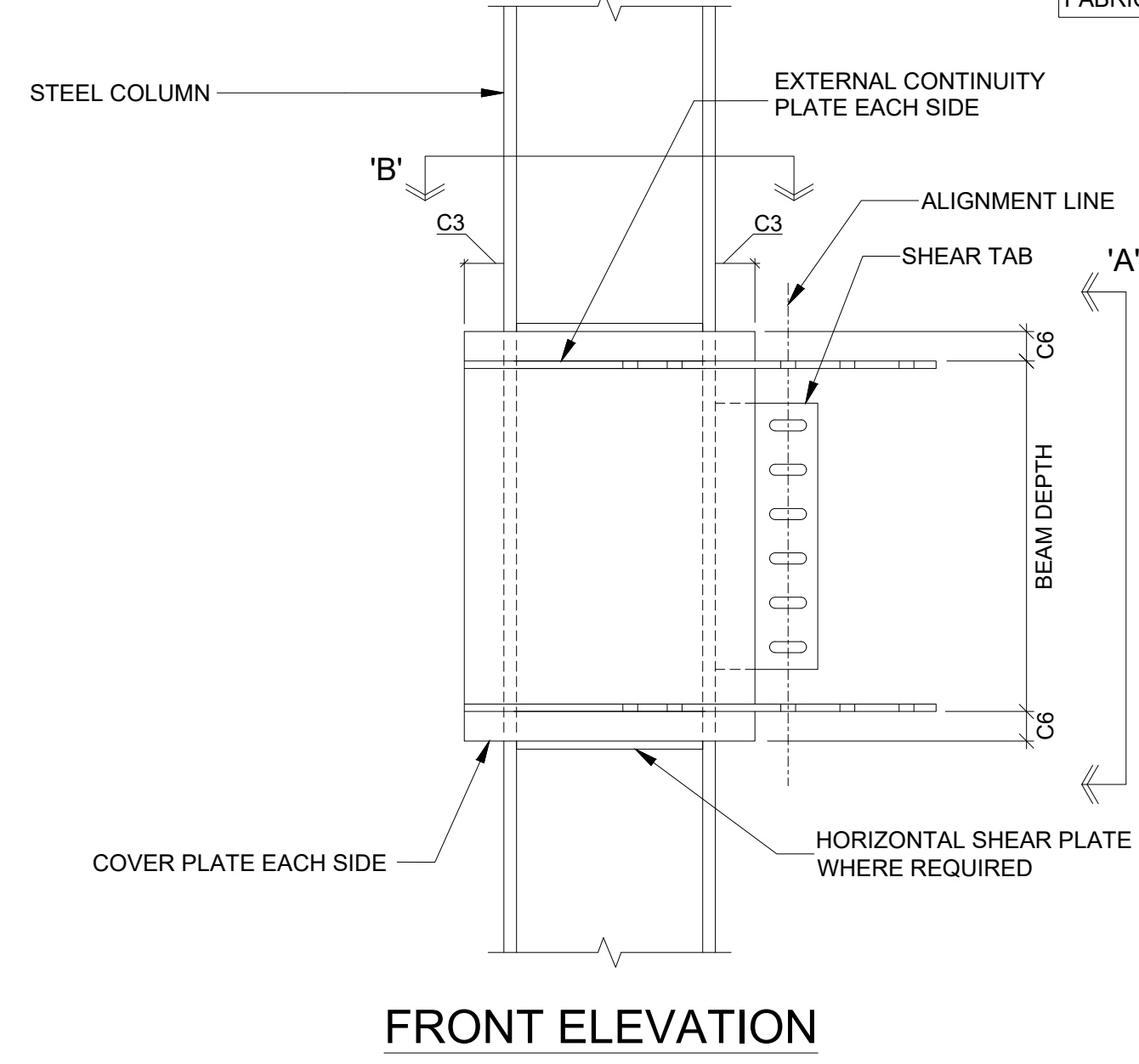
Fig. 23. Example TIO.

6 Checklist For Plan Reviewers and FAQ

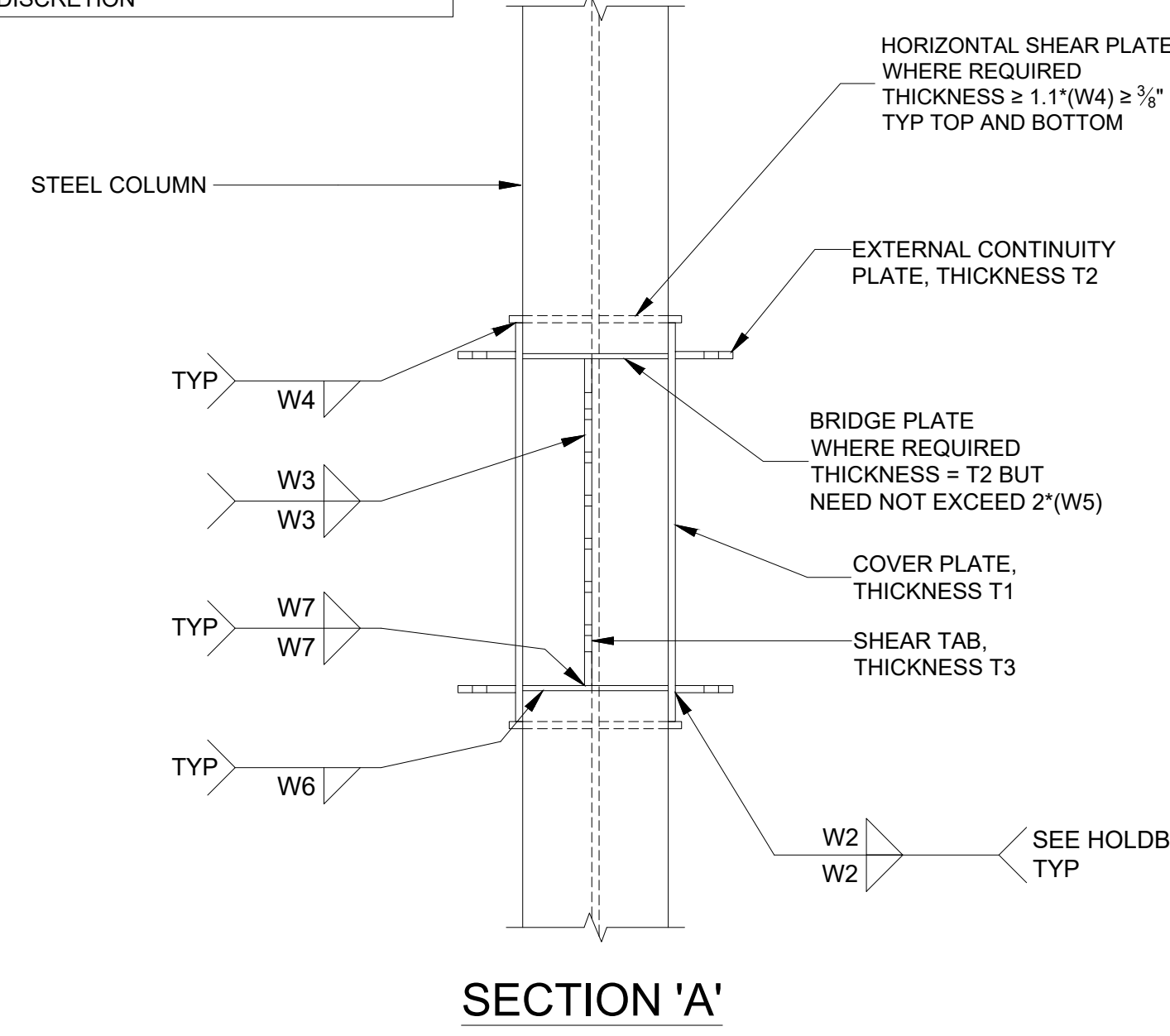
- 1) Are the maximum beam and column sizes acceptable?
 - a. Verify that the maximum beam depth is limited to a W40x section.
 - b. Verify that the maximum beam weight is limited to 232 lbs/ft for bolted top plates and 397lb/ft for welded top plates. Any deviation will be considered as an alternative system by HCAI.
 - c. Verify that the maximum rolled column depth is limited to W40x.
 - d. Verify that the maximum built-up column width is 30 inches and that its depth does not exceed the depth permitted for rolled wide-flange shapes.
- 2) Are the DuraFuse Frames connection designs acceptable?
 - a. For each connection, verify that the summary sheet indicates D/C ratios of 1.0 or less for all limit state checks, or that an OK is listed (for checks that do not have a D/C ratio).
 - b. Verify that the beam flange width-to-thickness ratio is not exceeding λ_p of $0.38 \cdot \sqrt{E/F_y}$ per AISC 360 Table B4.1b Case 11.
- 3) Have the beams been braced appropriately to achieve M_p beam?
 - a. Verify that the SEOR has provided lateral bracing per AISC 360.
 - b. Verify that lateral bracing does not exceed that indicated on DFF calculations.
- 4) Have the DFF connections been represented accurately in the computer model?
 - a. Verify that the beam-to-column connections are modeled as FR.
 - b. Verify that if the panel zones are being modeled as rigid, the calculations justify it.
 - c. If the models have non-prismatic beam sections, verify that the stiffness properties match those of the respective connection ID.
- 5) Are protected zones clearly marked?
 - a. Verify that the fuse plate yield regions are marked as protected zones.
 - b. Verify that the beam or other connection plates have not been marked as protected.
- 6) Are the gravity cantilever beams attached to the external continuity plates acceptable?
 - a. Verify that the details match those herein.
 - b. Verify that the cantilever loads have been included in connection calculations.
- 7) Are orthogonal drag beams passing through the DFF connection acceptable?
 - a. Verify that the drag loads have been included in connection calculations.
- 8) Has connection geometry been accounted for in architectural details?
 - a. Verify that the slab edge distance is sufficient to accommodate the connection by comparing the required slab edge with that provided.
 - b. Check perimeter frames and interior frames that are adjacent to openings.
- 9) Have the connections been checked for collector/drag loads?
 - a. Each required connection is designed for a minimum of $0.1A_gF_y$ (10%) beam capacity. Verify that the calculations are updated when collector diagram shows demand greater than 10% of beam capacity at a connection.
 - b. The top plates are designed to take all the collector/drag loads.
- 10) Is there a guide to understand the twenty (20) steps under calculation package?
 - a. Refer to AISC 358-22 Chapter 15 commentary which provides a detailed explanation on each step.

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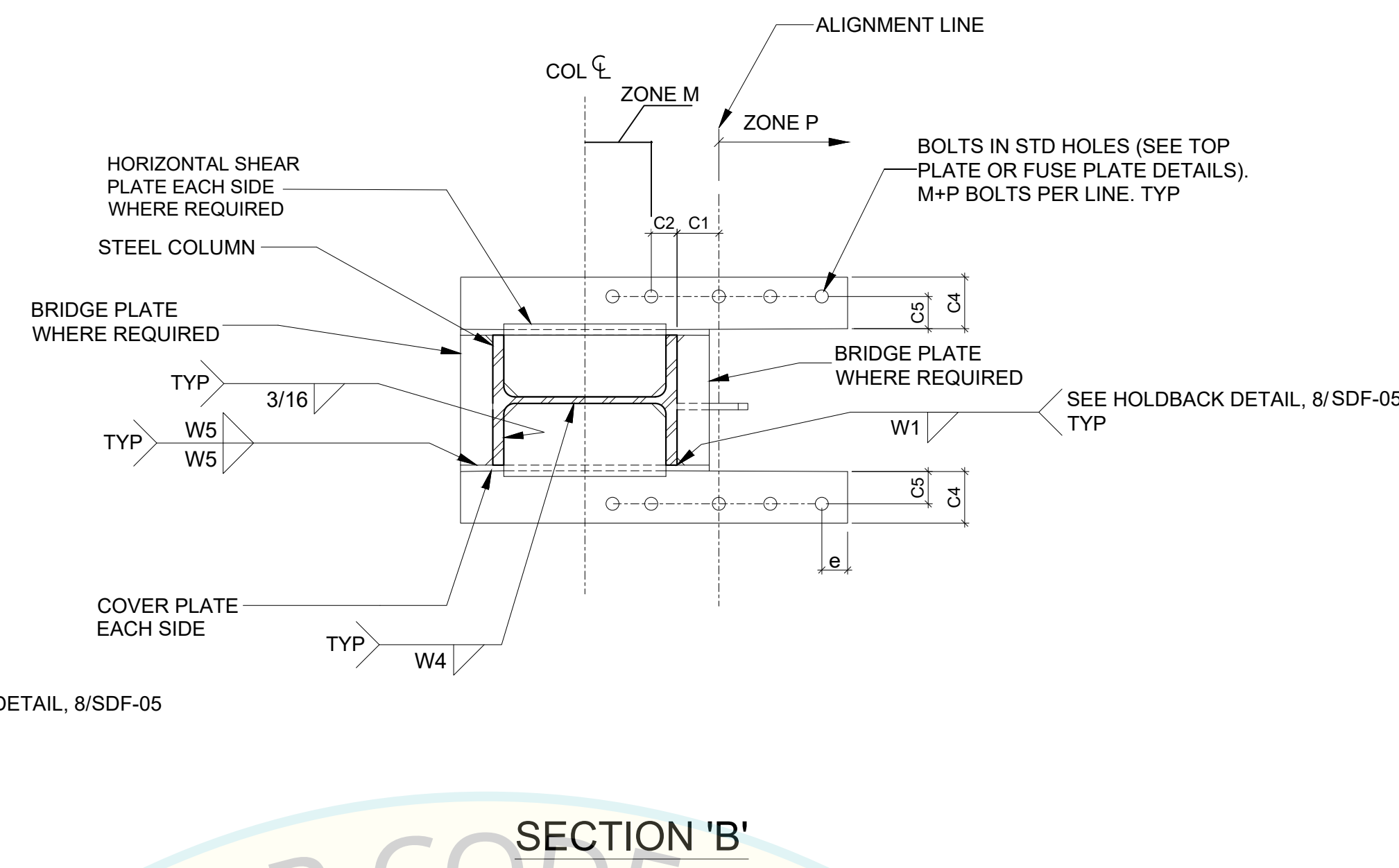
NOTES
 1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-02 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04
 2. WELDS W2, W3, W4, W5, W6, & W7 MAY BE SUBSTITUTED FOR AN EQUIVALENT PJP WELD AT FABRICATORS DISCRETION



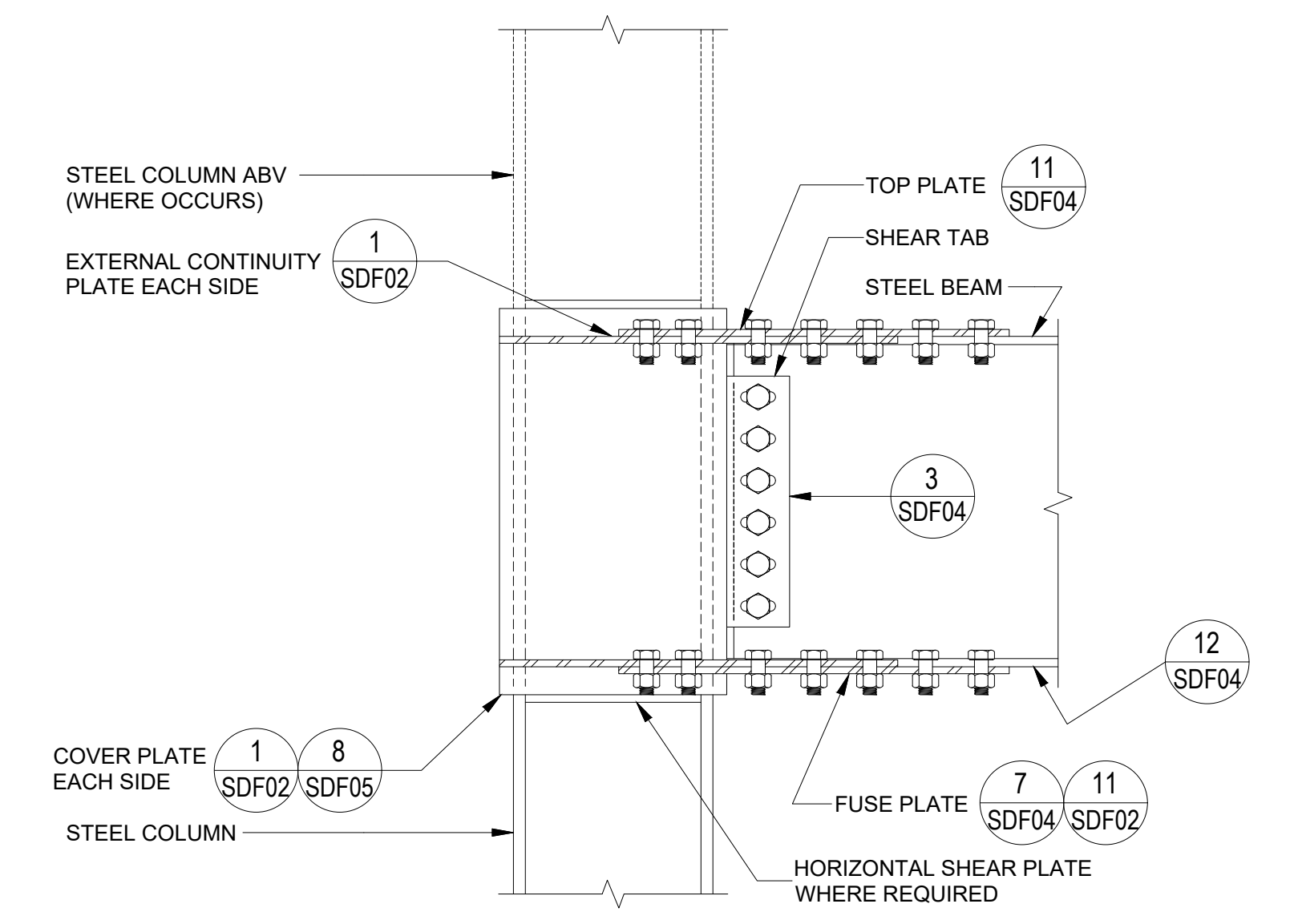
FRONT ELEVATION



SECTION 'A'



SECTION 'B'



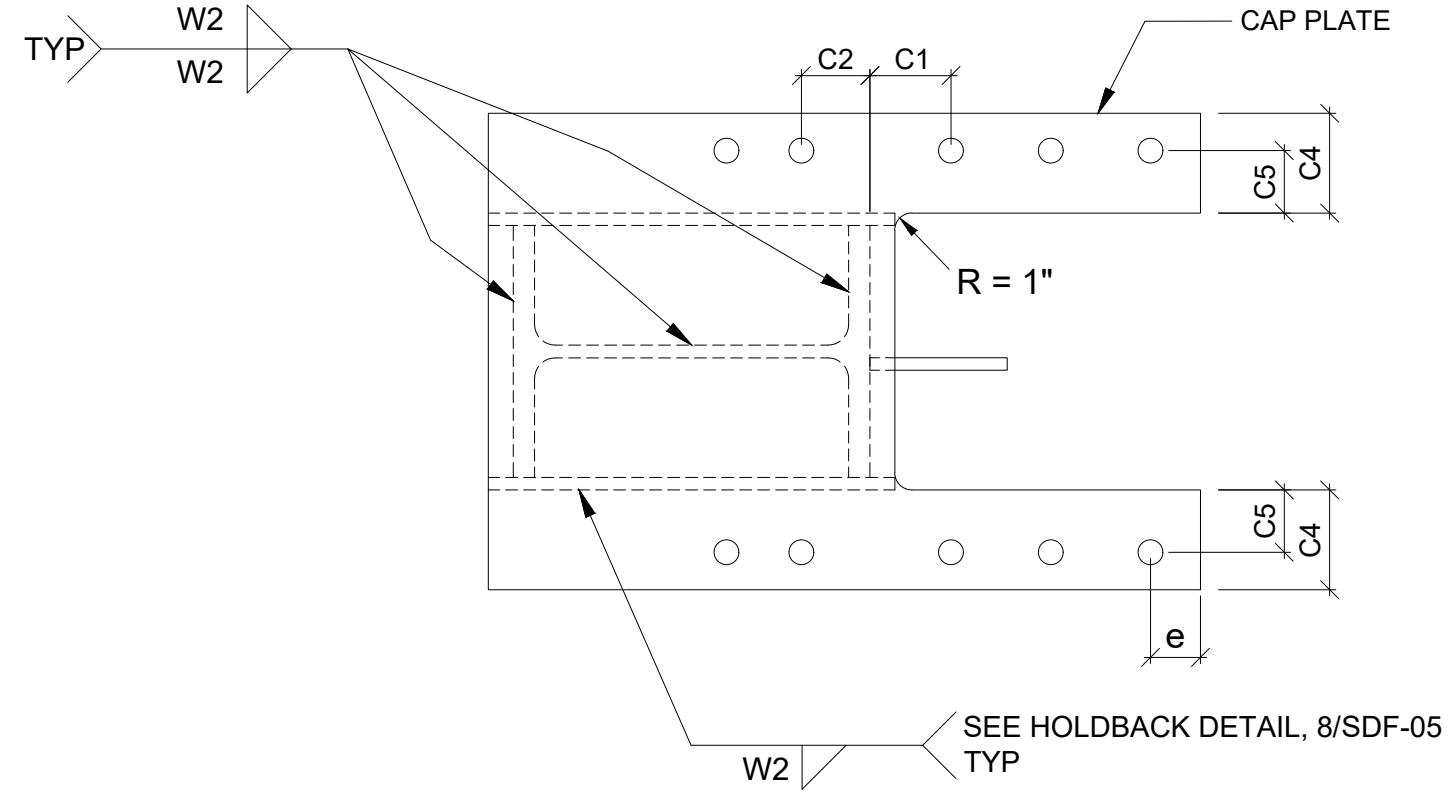
1 ONE SIDED CONNECTION - COLUMN ASSEMBLY
 SCALE: NTS

ID	MEMBER SIZES		NO DRAG PARAMETERS		
	COLUMN	BEAM	M BOT	P BOT	B BOT

NOTES:
 1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
 2. THE "M", "P", AND "B" BOLT COUNTS LISTED IN THIS TABLE ONLY APPLY TO THE FUSE PLATES, BOTTOM EXTERNAL CONTINUITY PLATES, AND BOTTOM FLANGE OF THE BEAM. SEE ONE AND TWO SIDED CONNECTION SCHEDULES FOR BOLT COUNTS ON TOP PLATES, TOP EXTERNAL CONTINUITY PLATES, AND TOP FLANGE OF THE BEAM, DETAIL 9 ON THIS SHEET.

4 ONE SIDED CONNECTION
 SCALE: NTS

NOTES
 1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-02 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04.
 2. CONNECTION PER 1/SDF-02 EXCEPT AT TOP FLANGE LEVEL AS SHOWN.
 3. SHEAR PLATES AND BRIDGE PLATES ARE NOT REQUIRED AT TOP FLANGE LEVEL.



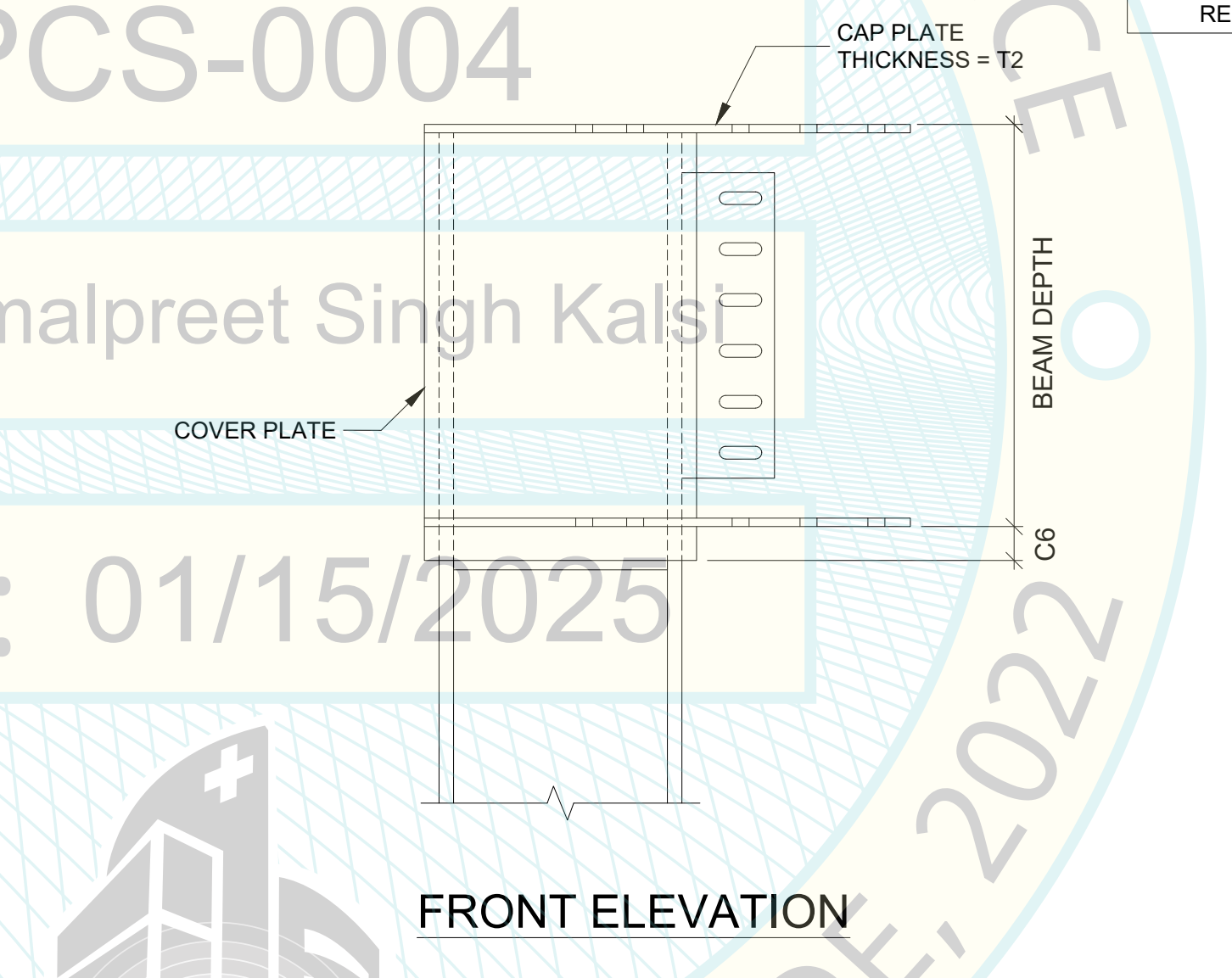
TOP VIEW

5 ONE SIDED BOTTOM BOLT CONNECTION SCHEDULE
 SCALE: NTS

ID	MEMBER SIZES		PLATE THICKNESS			DIMENSIONS					WELDS							BOLTS PER CONNECTION (EACH SIDE)						
	COLUMN	BEAM	T1	T2	T3	C1	C2	C3	C4	C5	C6	W1	W2	W3	W4	W5	W6	W7	DIA	SPACING	M	P	B	N

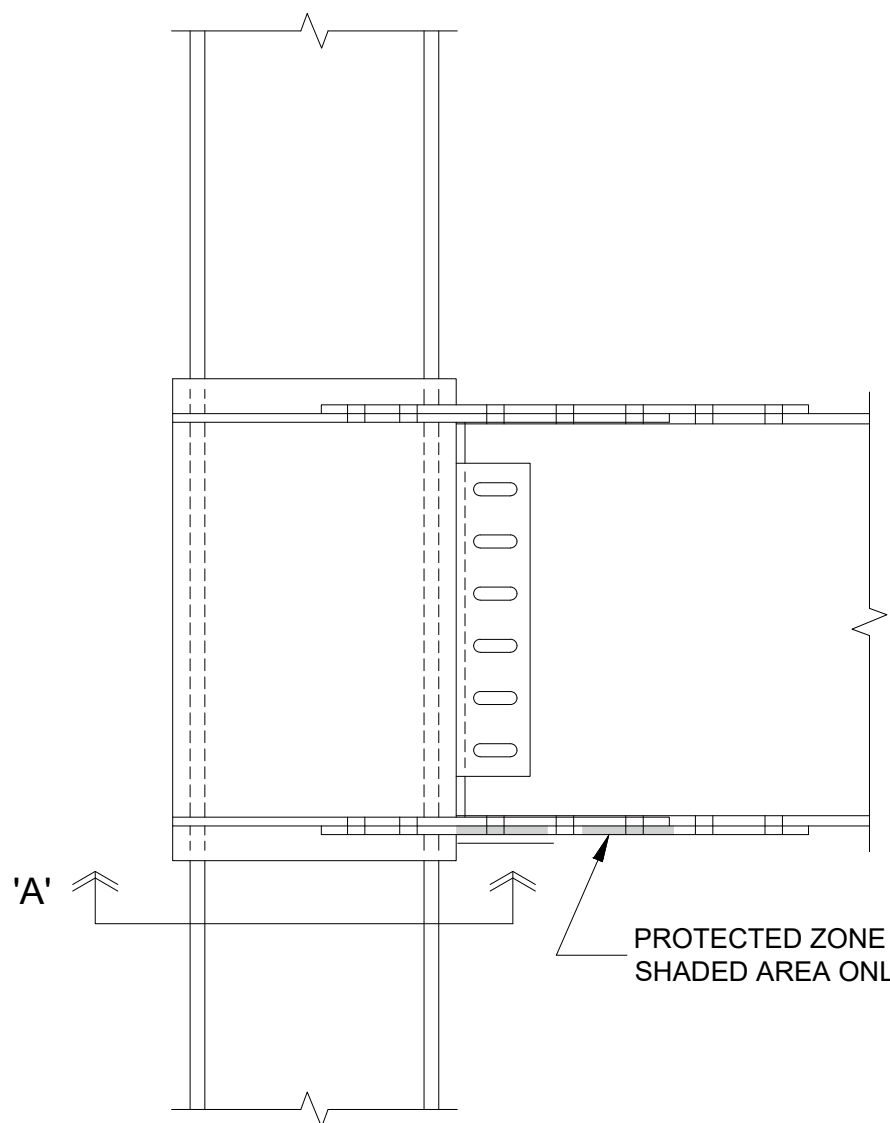
9 ONE SIDED CONNECTION SCHEDULE
 SCALE: NTS

7 TOP-OF-COLUMN CAP PLATE DETAIL FOR ONE SIDED CONNECTION
 SCALE: NTS



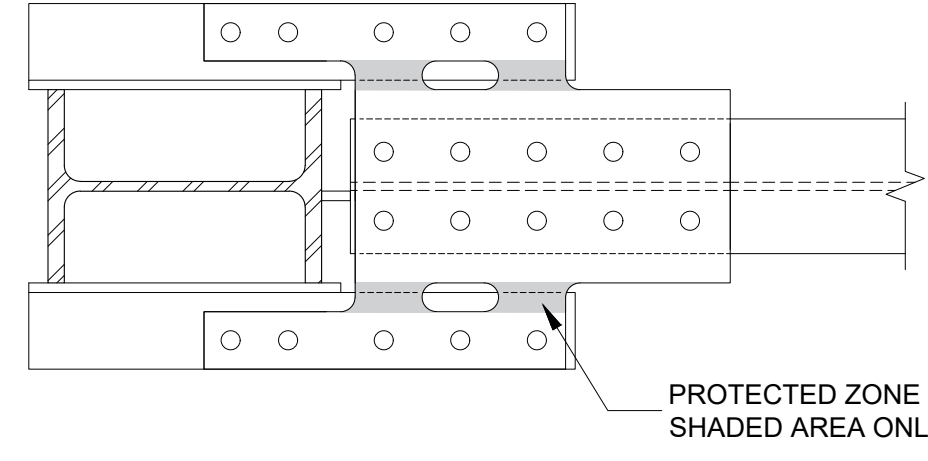
FRONT ELEVATION

NOTES
 1. THE SHADED AREA IS A PROTECTED ZONE. NO OTHER COMPONENT OF THE CONNECTION, NOR ANY PORTION OF THE BEAMS OR COLUMN ARE A PROTECTED ZONE.
 2. PROTECTED ZONE SHALL BE MARKED USING ANY NON-DSTRUCTIVE METHODS PRIOR TO AND AFTER APPLICATION OF SPRAYED ON FIRE-RESISTING MATERIAL.
 3. AREA MARKED AS PROTECTED ZONE SHALL COMPLY WITH THE FOLLOWING REQUIREMENTS:
 3.1. WITHIN THE PROTECTED ZONE, DISCONTINUITIES CREATED BY FABRICATION OR ERECTION OPERATIONS, BEYOND THOSE REQUIRED FOR INITIAL FABRICATION OF THE PLATE, SUCH AS TACK WELDS, ERECTION AIDS, AIR-ARC GOUGING AND THERMAL CUTTING SHALL BE REPAIRED. REPAIR METHOD SHALL BE APPROVED BY THE ENGINEER OF RECORD.
 3.2. WELDING, BOLTING, SCREWING OR SHOT-IN ATTACHMENTS FOR PARTITIONS, DUCT WORK, PIPING OR OTHER CONSTRUCTION SHALL NOT BE PLACED WITHIN THE PROTECTED ZONE.



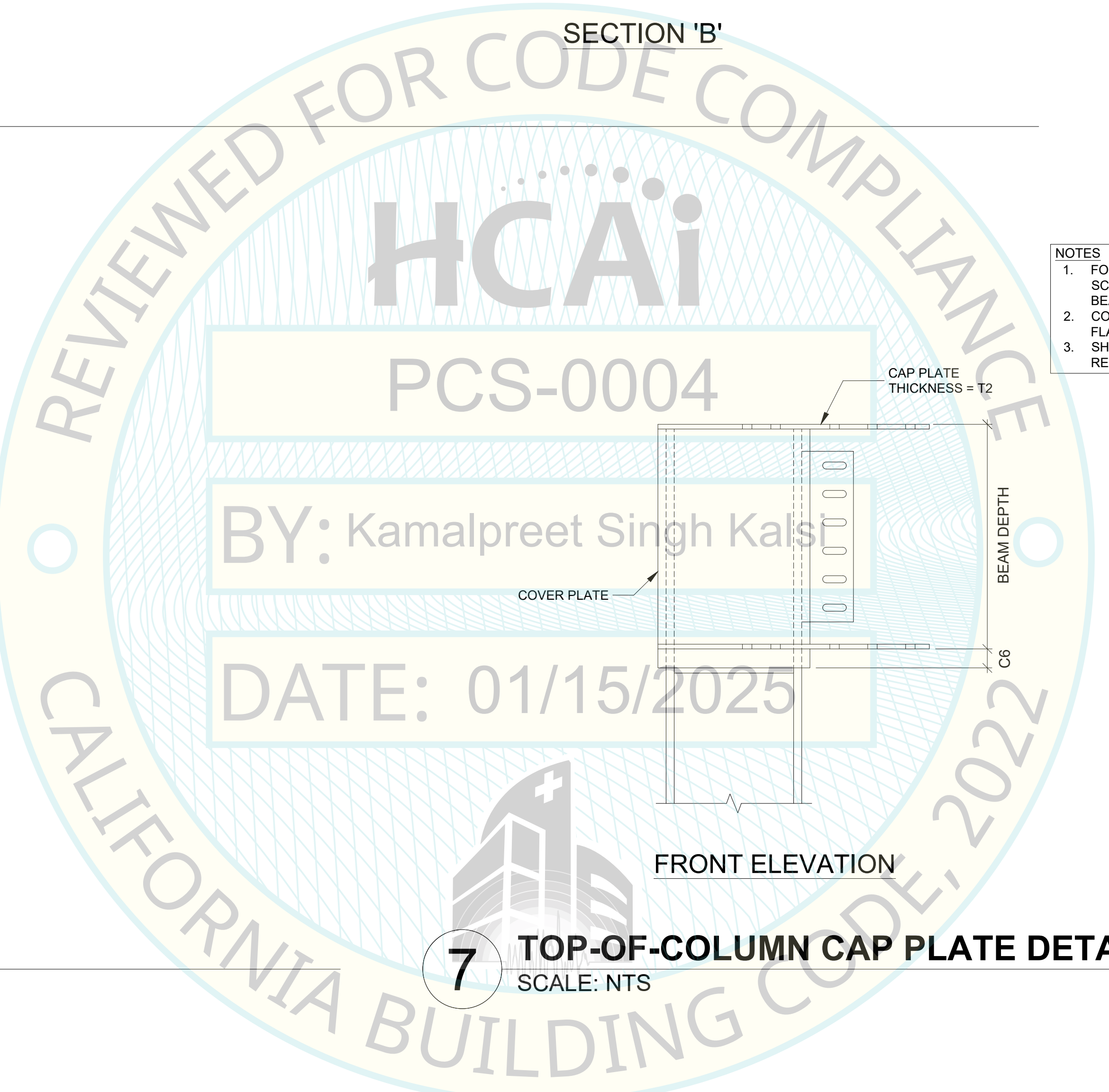
SECTION 'A'

11 ONE SIDED PROTECTED ZONE
 SCALE: NTS



SECTION 'A'

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PROJECT: HCAI CALCULATIONS

XXXX, XX

DESCRIPTION	DATE	NO.	ADDITIONAL NOTES:
			HCAI

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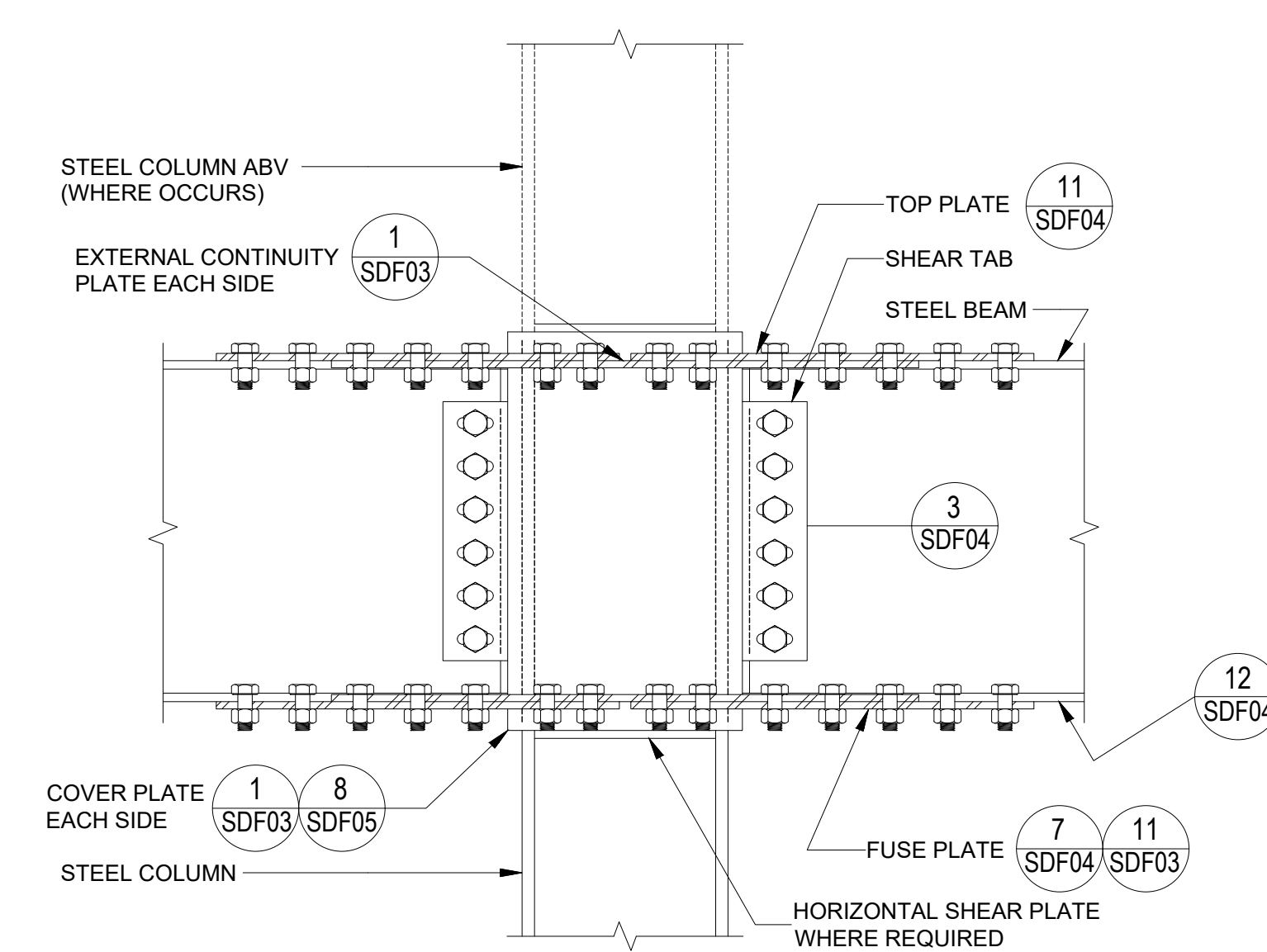
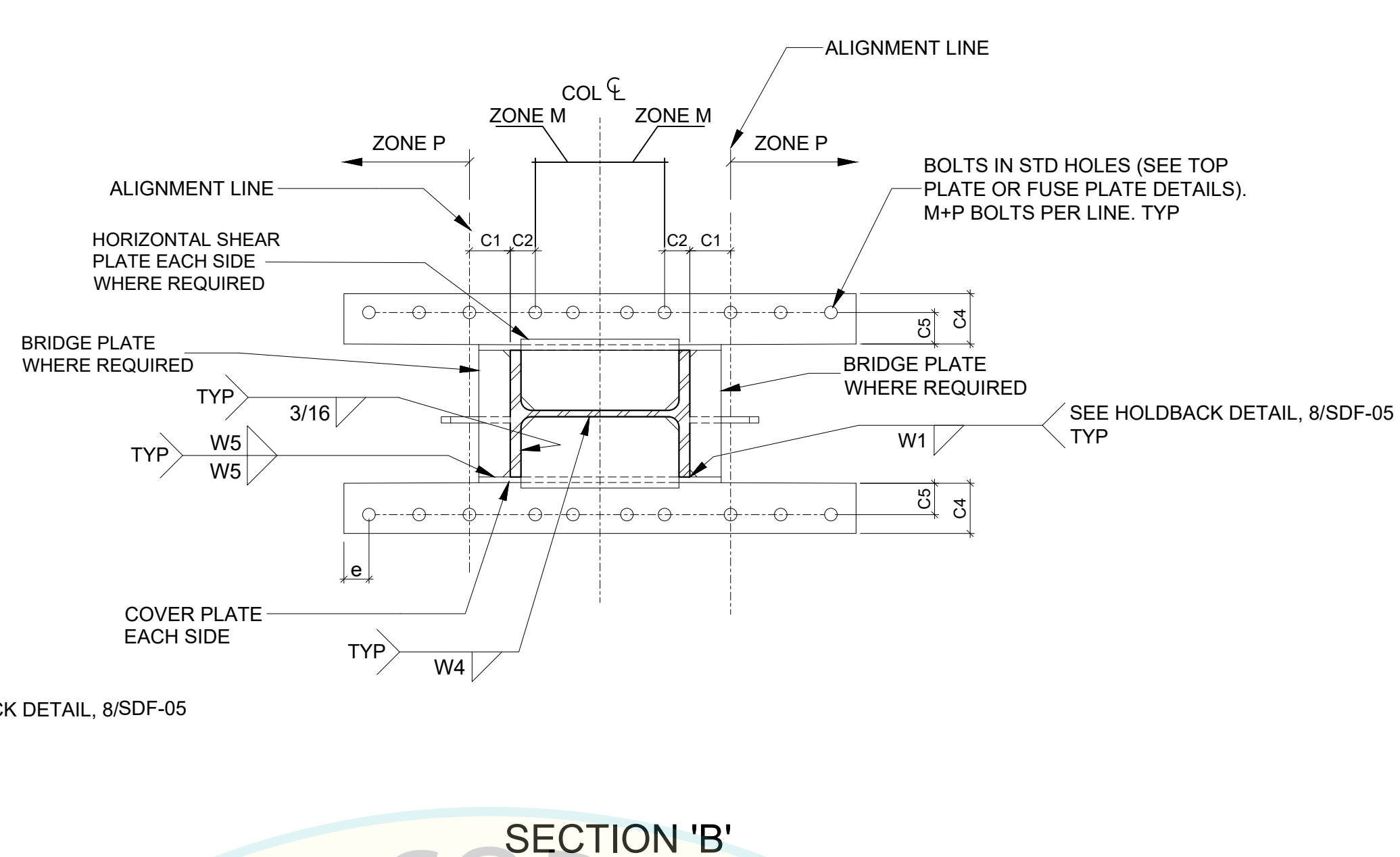
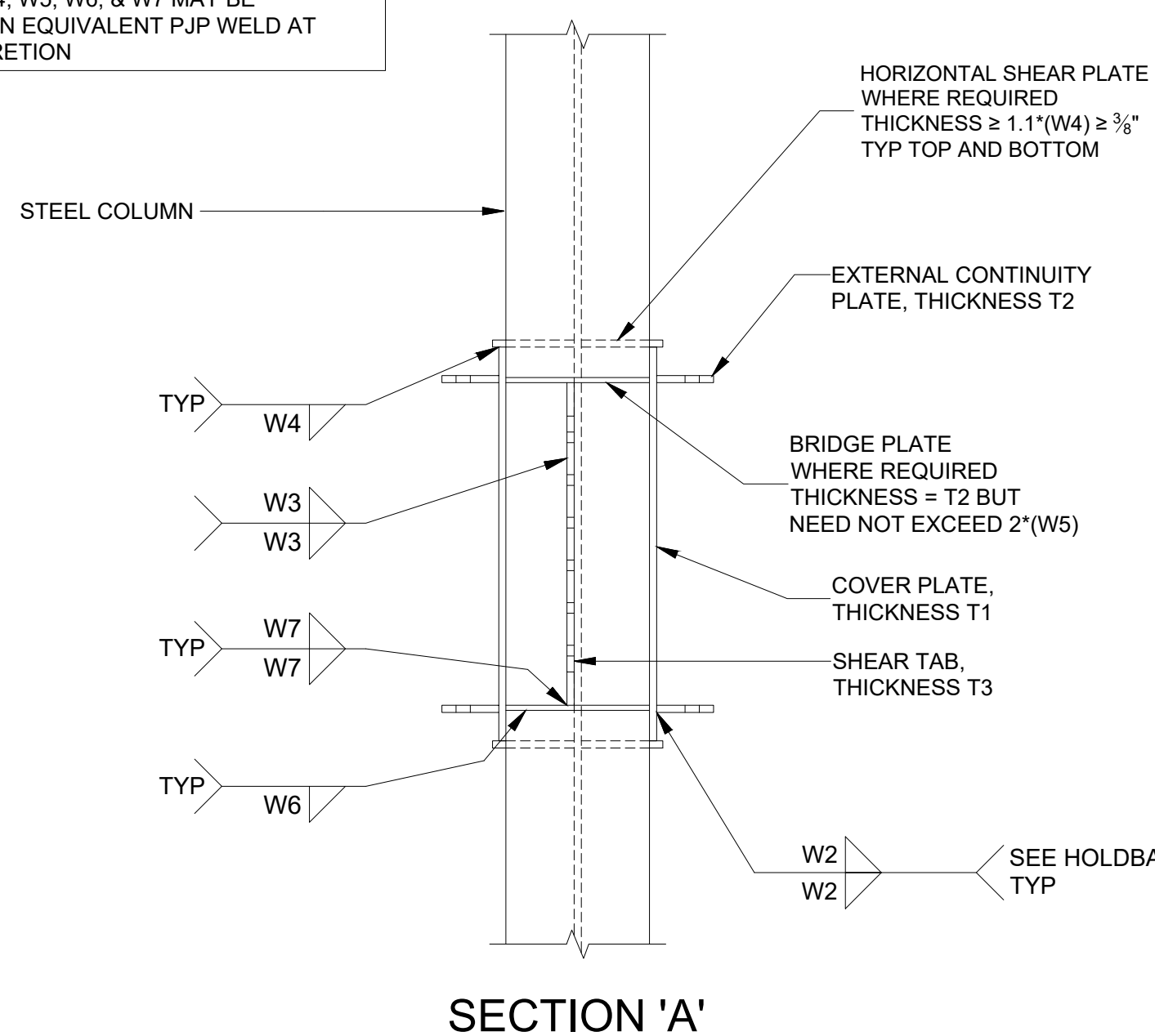
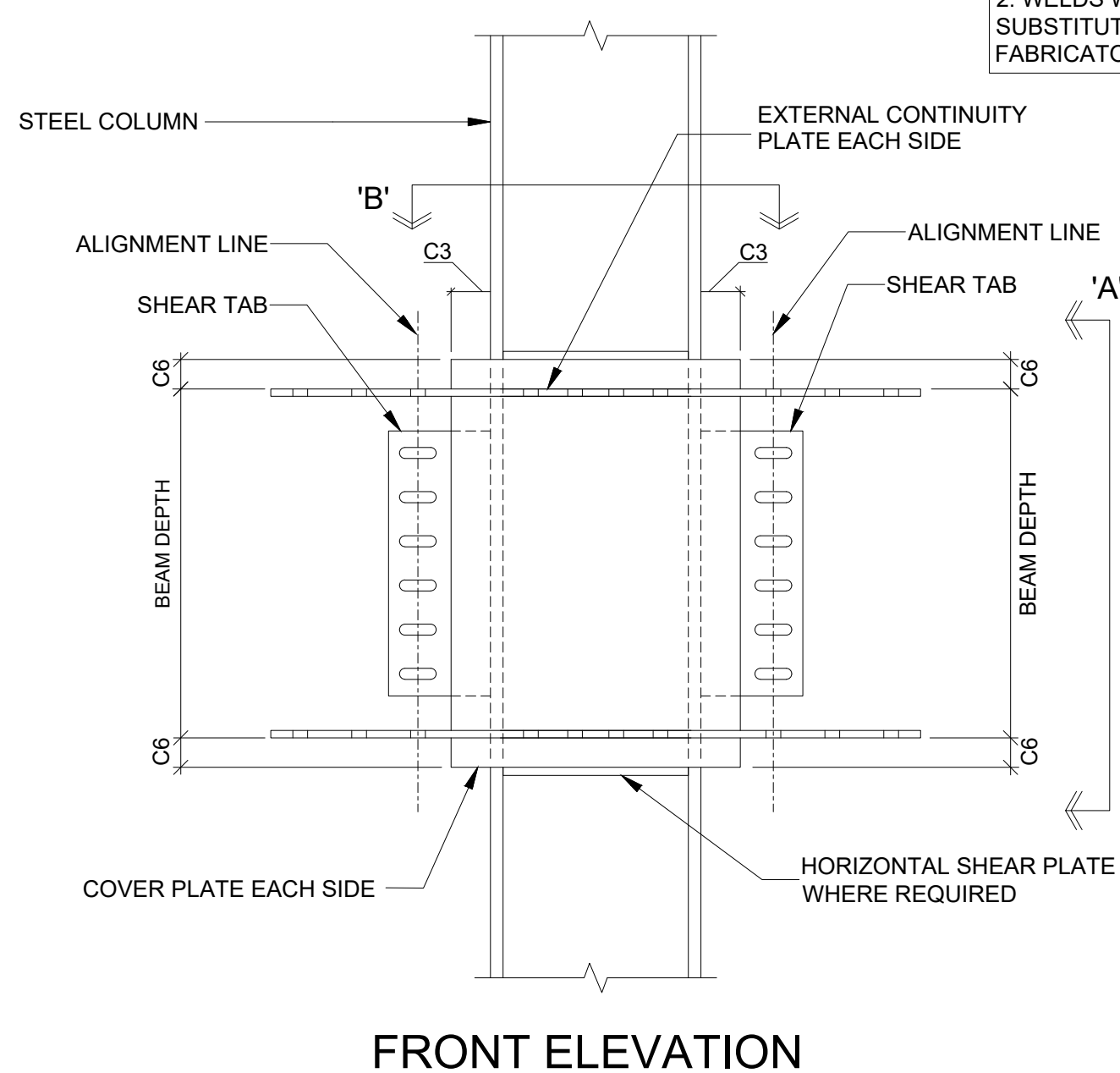
SHEET TITLE: ONE-SIDED CONNECTION DETAILS

DRAWN BY: AM	DATE: 12/19/2024
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00
SHEET NUMBER:	

SDF-02

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NOTES
 1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL, 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL, 9/SDF-04
 2. WELDS W2, W3, W4, W5, W6, & W7 MAY BE SUBSTITUTED FOR AN EQUIVALENT PJP WELD AT FABRICATOR'S DISCRETION



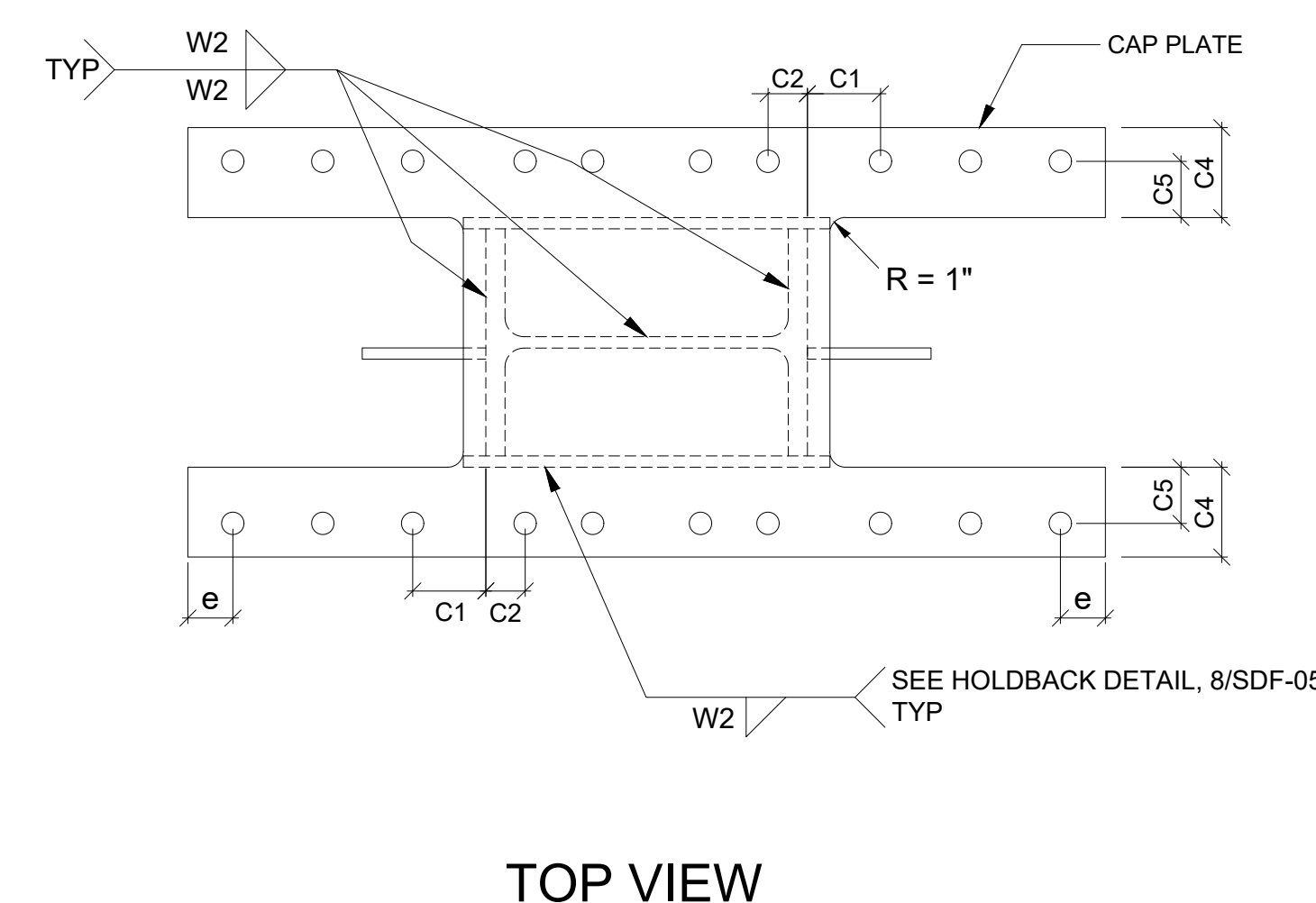
1 TWO SIDED CONNECTION - COLUMN ASSEMBLY
 SCALE: NTS

ID	MEMBER SIZES		NO DRAG PARAMETERS		
	COLUMN	BEAM	M BOT	P BOT	B BOT

NOTES:
 1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
 2. THE "M," "P," AND "B" BOLT COUNTS LISTED IN THIS TABLE ONLY APPLY TO THE FUSE PLATES, BOTTOM EXTERNAL CONTINUITY PLATES, AND BOTTOM FLANGE OF THE BEAM. SEE ONE AND TWO SIDED CONNECTION SCHEDULES FOR BOLT COUNTS ON TOP PLATES, TOP EXTERNAL CONTINUITY PLATES, AND TOP FLANGE OF THE BEAM. DETAIL 9 ON THIS SHEET.

4 TWO SIDED CONNECTION
 SCALE: NTS

NOTES
 1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04.
 2. CONNECTION PER 1/SDF-03 EXCEPT AT TOP FLANGE LEVEL AS SHOWN.
 3. SHEAR PLATES AND BRIDGE PLATES ARE NOT REQUIRED AT TOP FLANGE LEVEL.

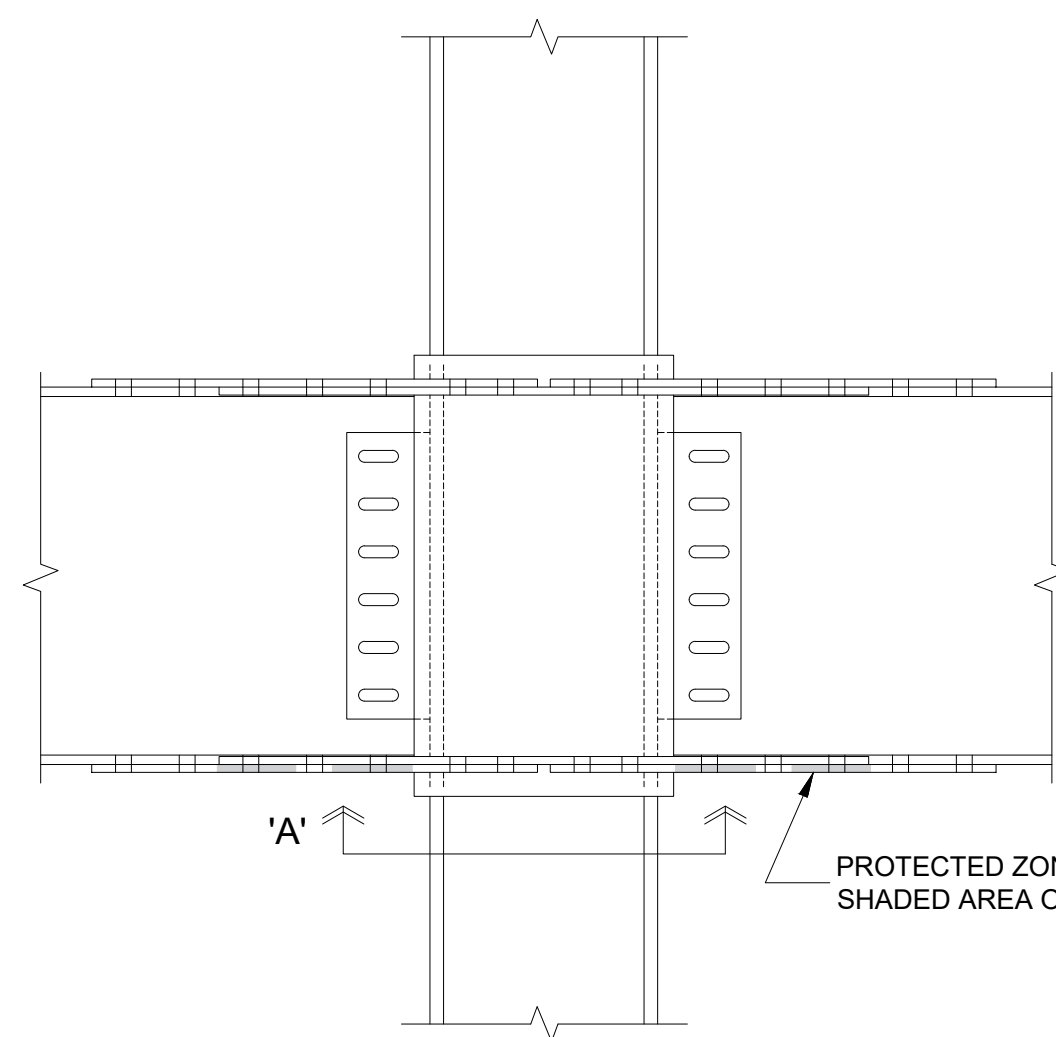


5 TWO SIDED BOTTOM BOLT CONNECTION SCHEDULE
 SCALE: NTS

7 TOP-OF-COLUMN DETAIL FOR TWO SIDED CONNECTION
 SCALE: NTS

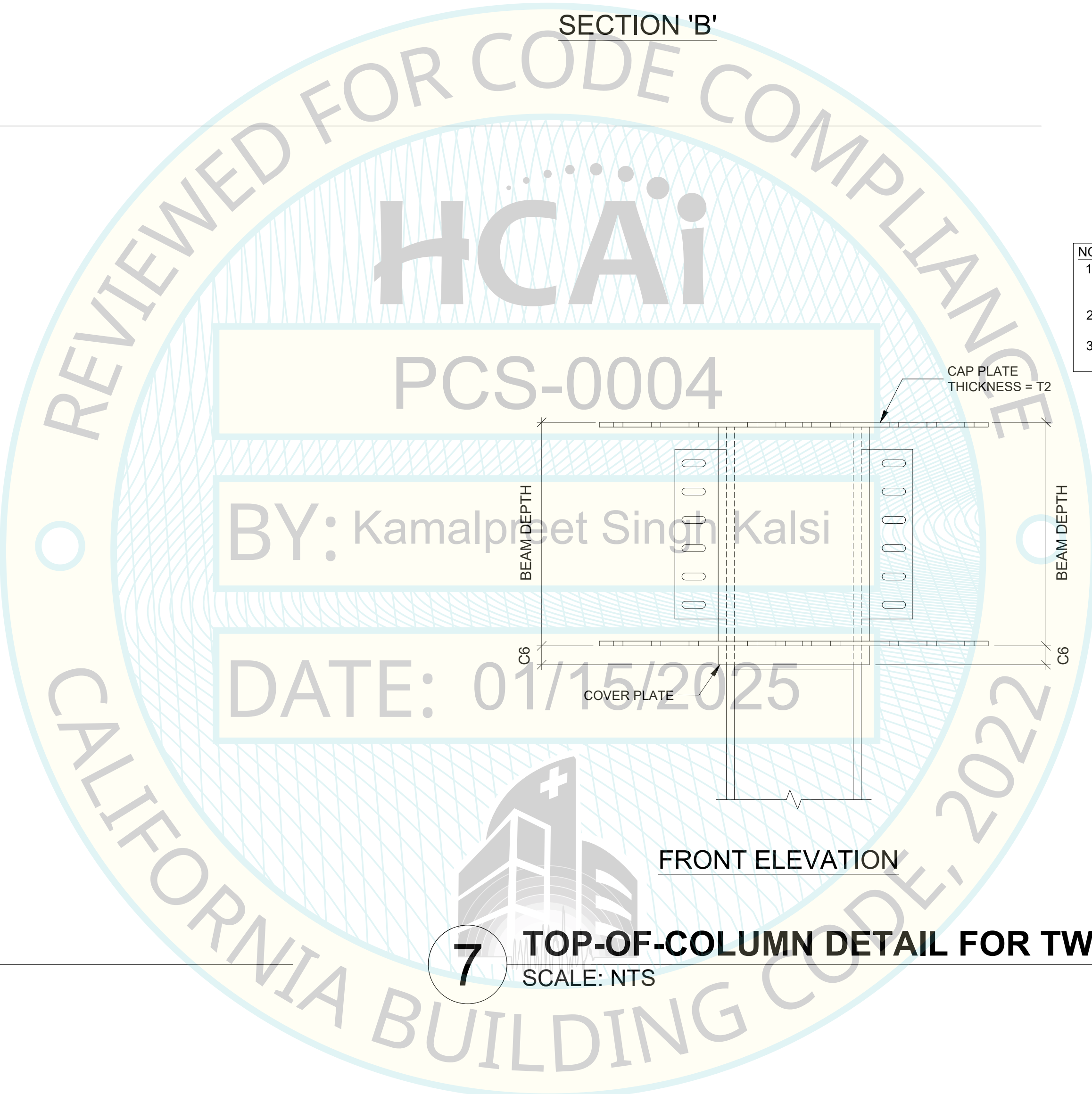
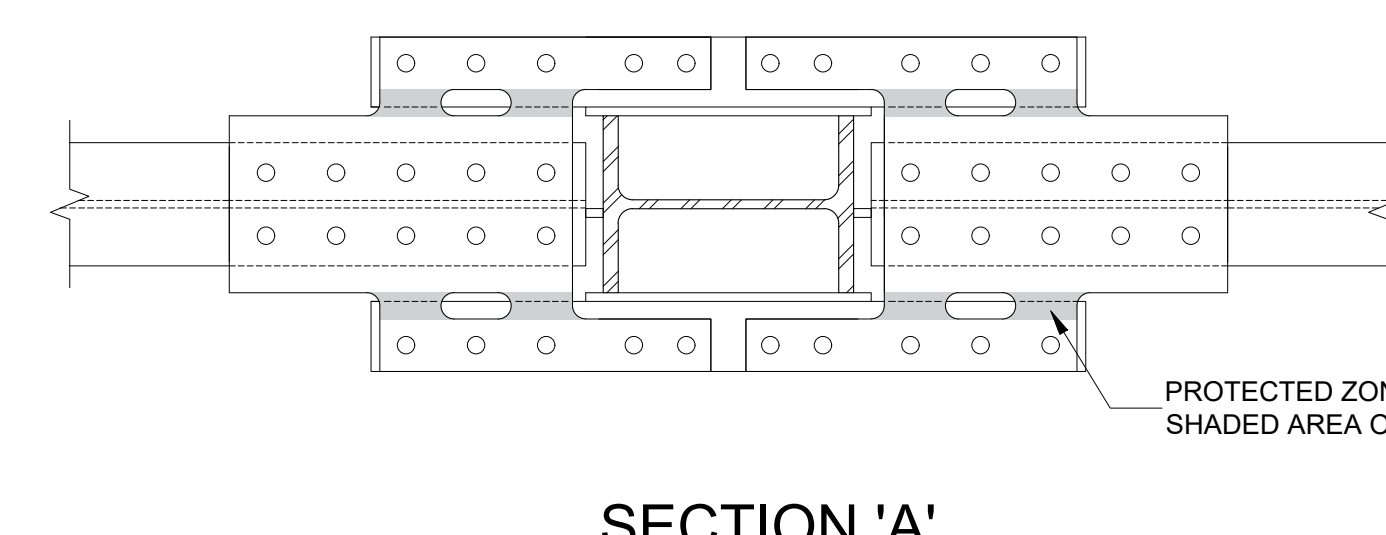
ID	MEMBER SIZES		PLATE THICKNESS			DIMENSIONS					WELDS							BOLTS PER CONNECTION (EACH SIDE)						
	COLUMN	BEAM	T1	T2	T3	C1	C2	C3	C4	C5	C6	W1	W2	W3	W4	W5	W6	W7	DIA	SPACING	M	P	B	N

9 TWO SIDED CONNECTION SCHEDULE
 SCALE: NTS



NOTES
 1. THE SHADED AREA IS A PROTECTED ZONE. NO OTHER COMPONENT OF THE CONNECTION, NOR ANY PORTION OF THE BEAMS OR COLUMN ARE A PROTECTED ZONE.
 2. PROTECTED ZONE SHALL BE MARKED USING ANY NON-DESTRUCTIVE METHODS PRIOR TO AND AFTER APPLICATION OF SPRAYED ON FIRE-RESISTING MATERIAL.
 3. AREA MARKED AS PROTECTED ZONE SHALL COMPLY WITH THE FOLLOWING REQUIREMENTS:
 3.1. WITHIN THE PROTECTED ZONE, DISCONTINUITIES CREATED BY FABRICATION OR ERECTION OPERATIONS, BEYOND THOSE REQUIRED FOR INITIAL FABRICATION OF THE PLATE, SUCH AS TACK WELDS, ERECTION AIDS, AIR-ARC GOUGING AND THERMAL CUTTING SHALL BE REPAIRED. REPAIR METHOD SHALL BE APPROVED BY THE ENGINEER OF RECORD.
 3.2. WELDING, BOLTING, SCREWING OR SHOT-IN ATTACHMENTS FOR PARTITIONS, DUCT WORK, PIPING OR OTHER CONSTRUCTION SHALL NOT BE PLACED WITHIN THE PROTECTED ZONE.

11 TWO SIDED PROTECTED ZONE
 SCALE: NTS



PROJECT:
HCAI CALCULATIONS

XXXX, XX

DESCRIPTION	DATE	NO.	ADDITIONAL NOTES:
			HCAI

STAMP AND SIGNATURE:

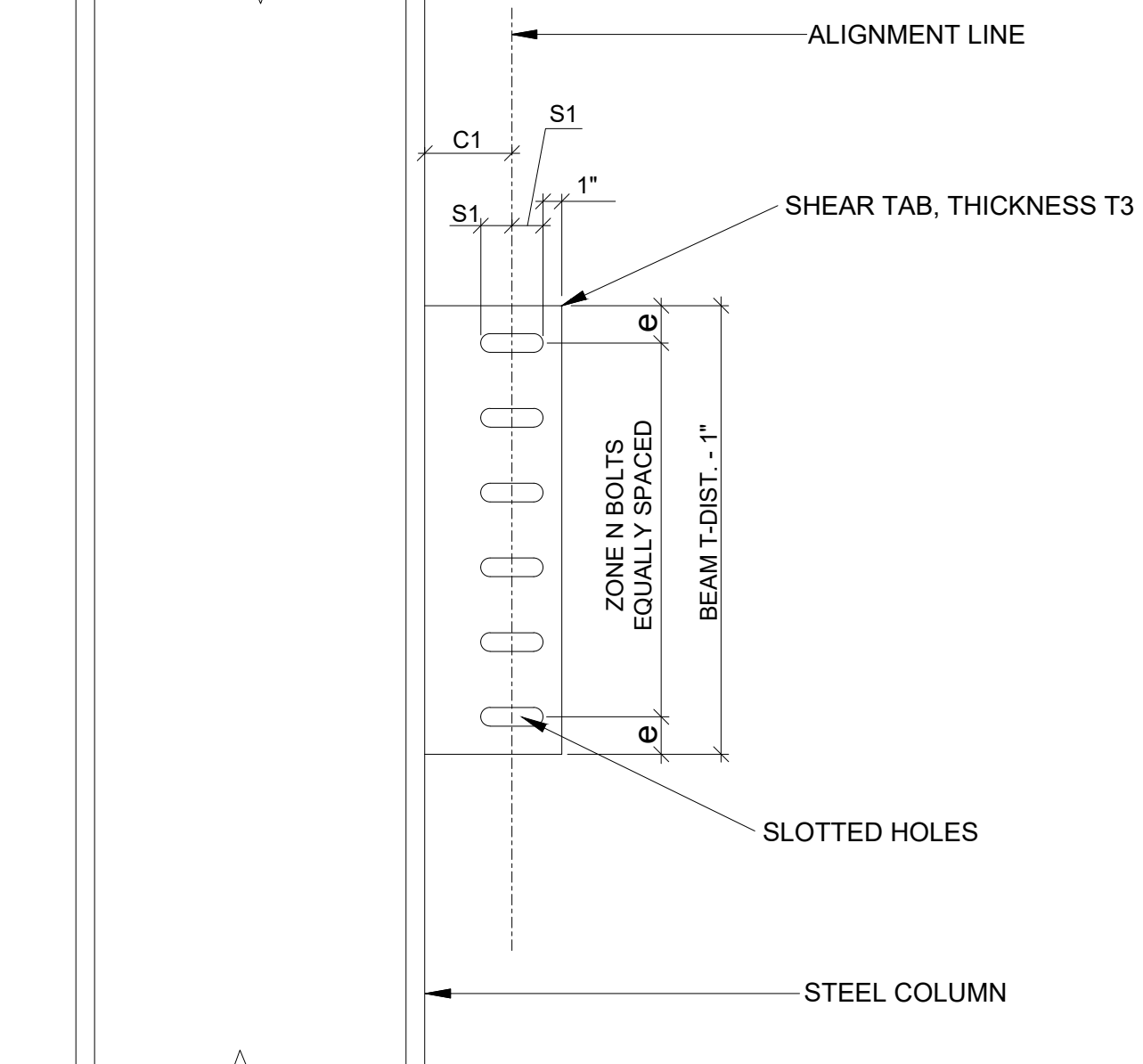


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SHEET TITLE:
TWO-SIDED CONNECTION DETAILS

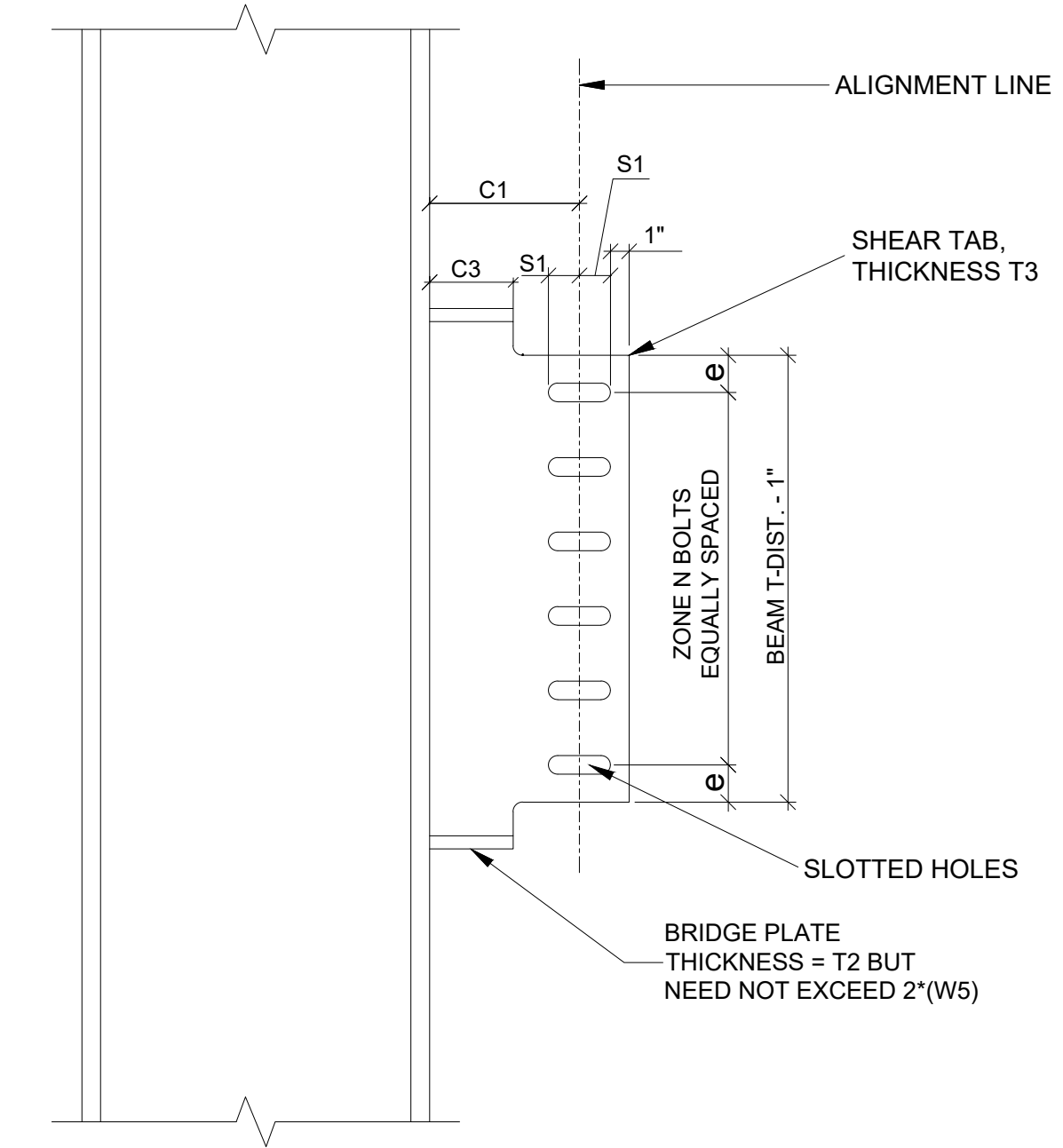
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NOTES
 1. FOR DIMENSIONS INDICATED IN TYPICAL DETAILS THIS SHEET, SEE CONNECTION SCHEDULE DETAILS 9/SDF-02 AND 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04

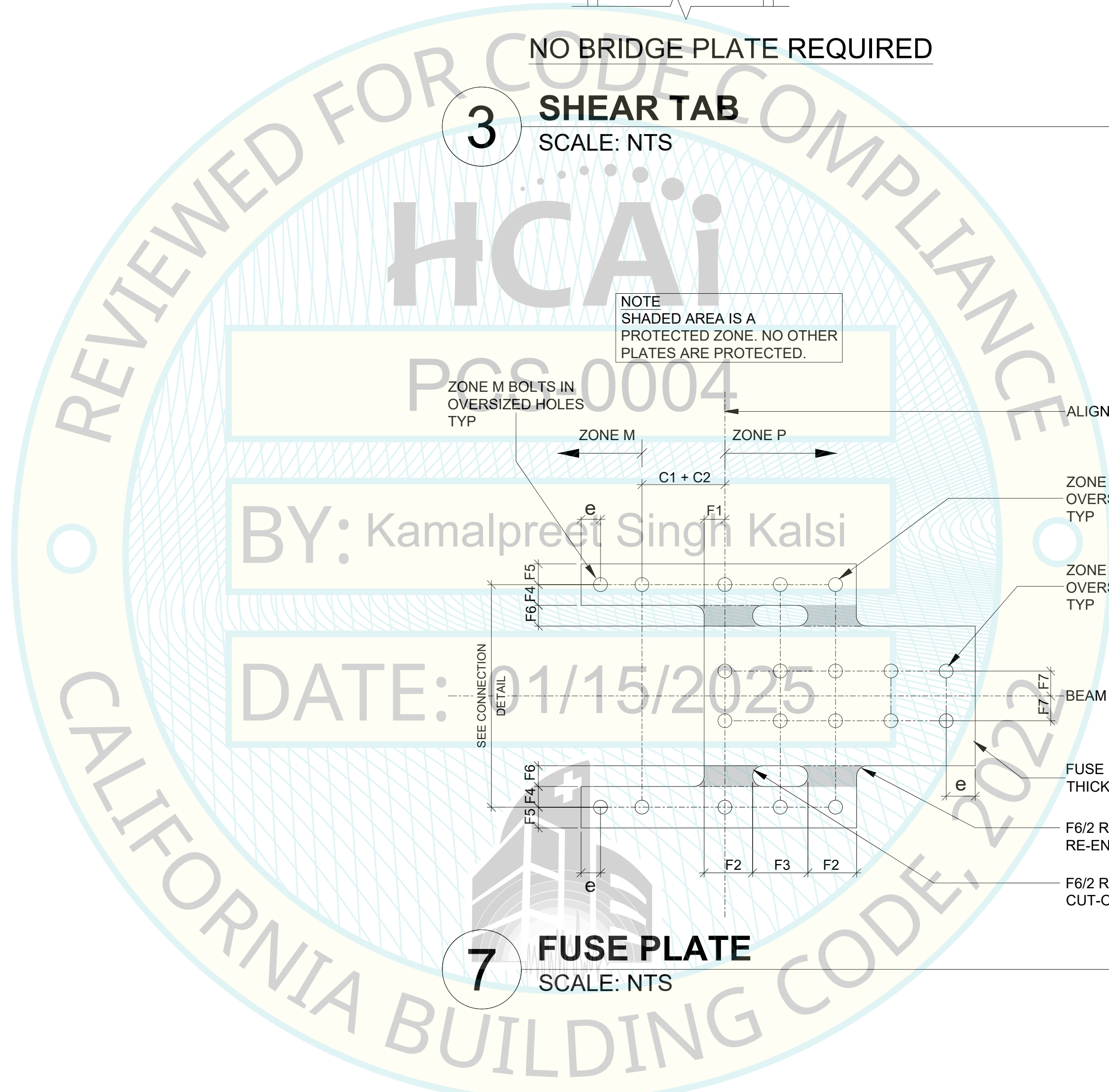


NO BRIDGE PLATE REQUIRED

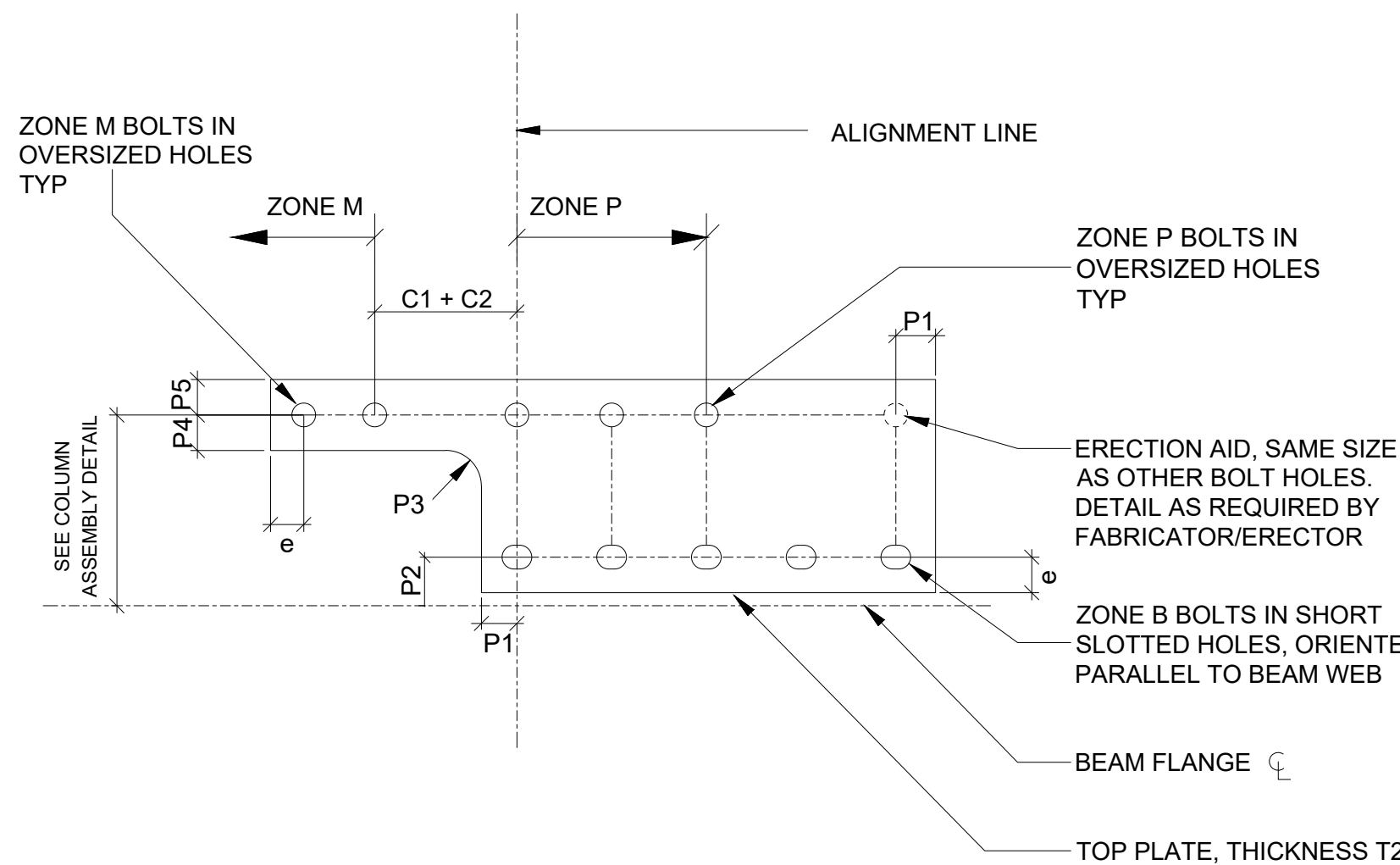
3 SHEAR TAB
SCALE: NTS



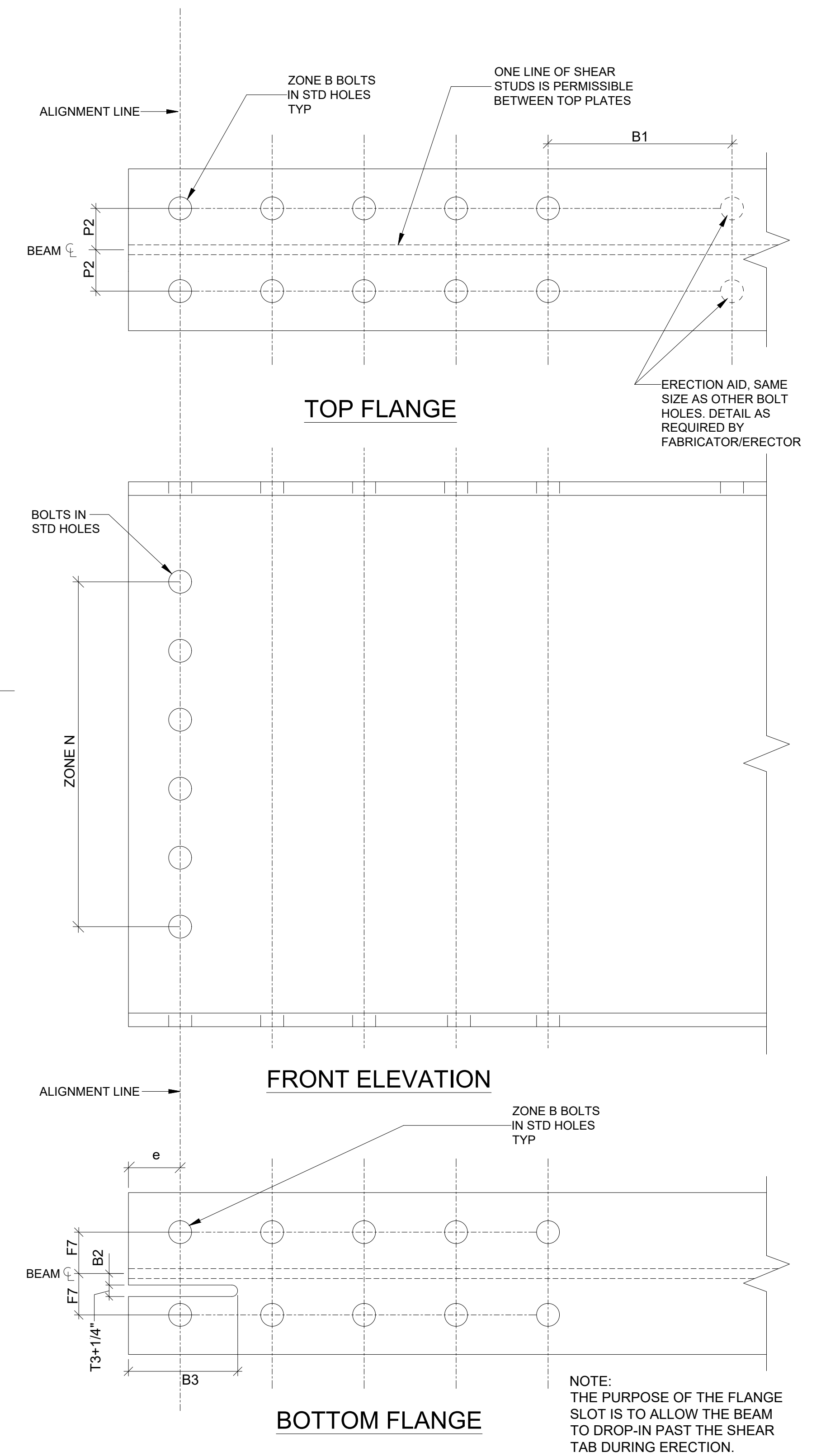
WHEN BRIDGE PLATE REQUIRED



7 FUSE PLATE
SCALE: NTS



11 TOP PLATE
SCALE: NTS



12 BEAM END
SCALE: NTS

DURAFUSE DRAWING VERSION: 1.5.48

ID	MEMBER SIZES		TOP PLATE DIMENSIONS					FUSE PLATE DIMENSIONS					SHEAR TAB	BEAM DIMENSIONS				
	COLUMN	BEAM	P1	P2	P3	P4	P5	F1	F2	F3	F4	F5		F6	F7	S1	B1	B2

NOTES:
 1. SEE CONNECTION SCHEDULE FOR C1, C2, T1, T2, T3, AND NUMBER OF BOLTS IN M, N, P, AND B ZONES.
 2. $e \ge 1-1/8"$ FOR 3/4" BOLTS, $1-1/4"$ FOR 7/8" BOLTS, $1-3/8"$ FOR 1" BOLTS, $1-5/8"$ FOR 1-1/8" BOLTS, OR $1-3/4"$ FOR 1-1/4" BOLTS.
 3. IF B2 AND B3 EQUAL ZERO THEN THERE IS NO SLOT IN THE BEAM FLANGE, AND THE BEAM CAN SWING AROUND THE SHEAR TAB DURING ERECTION.

9 PLATE AND BEAM END SCHEDULE
SCALE: NTS

PROJECT: HCAI CALCULATIONS

XXXX, XX

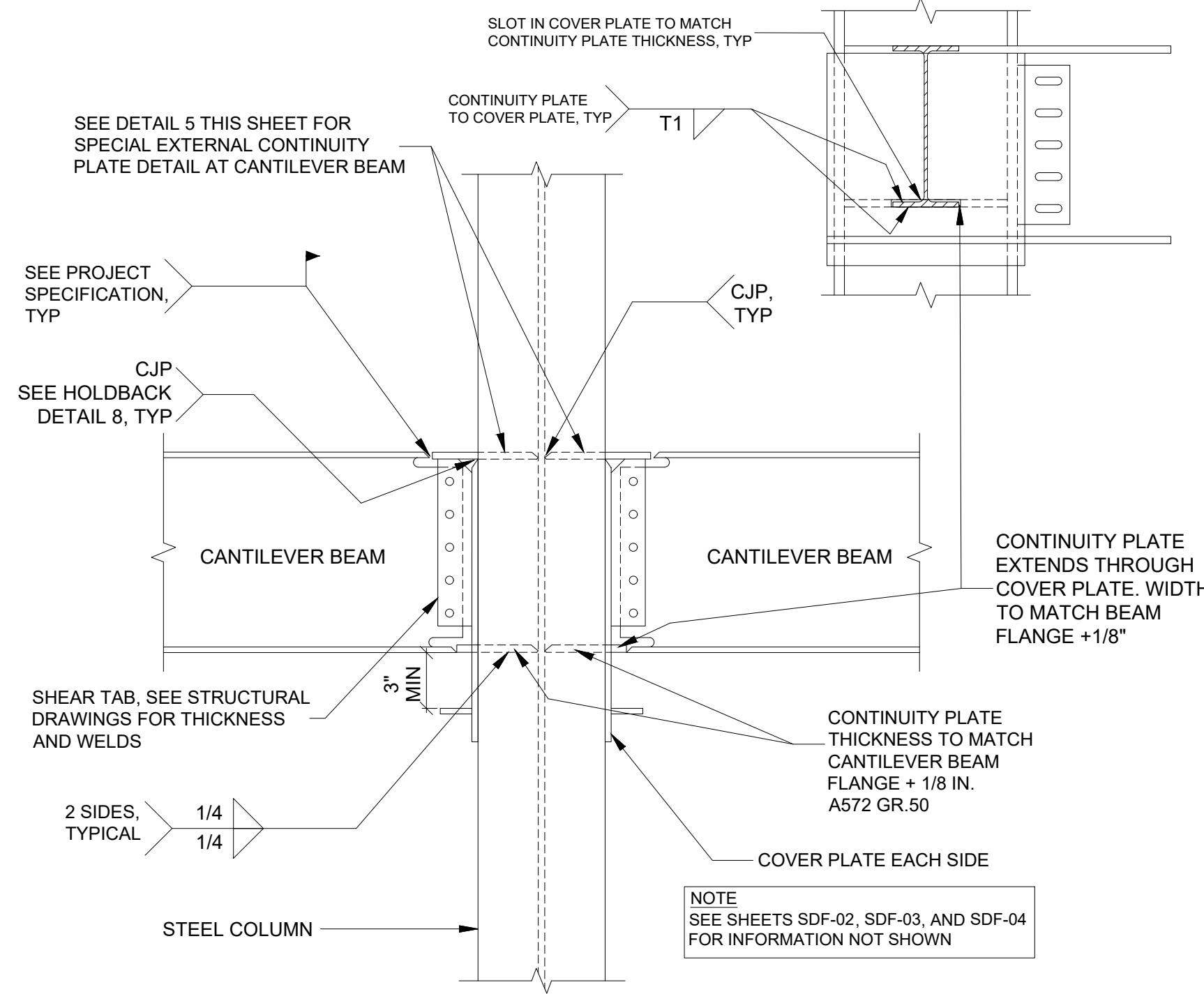
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ADDITIONAL NOTES:
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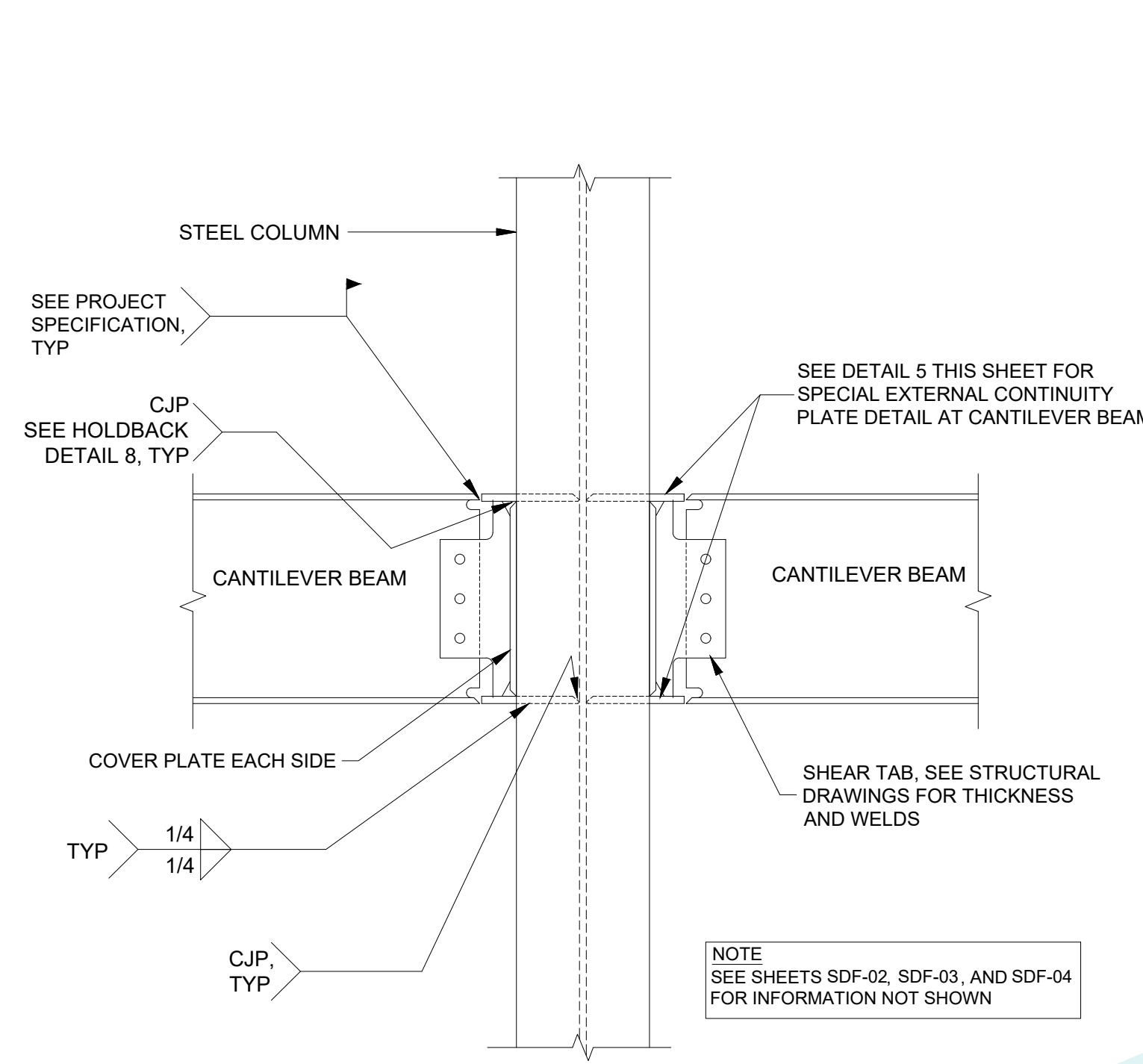
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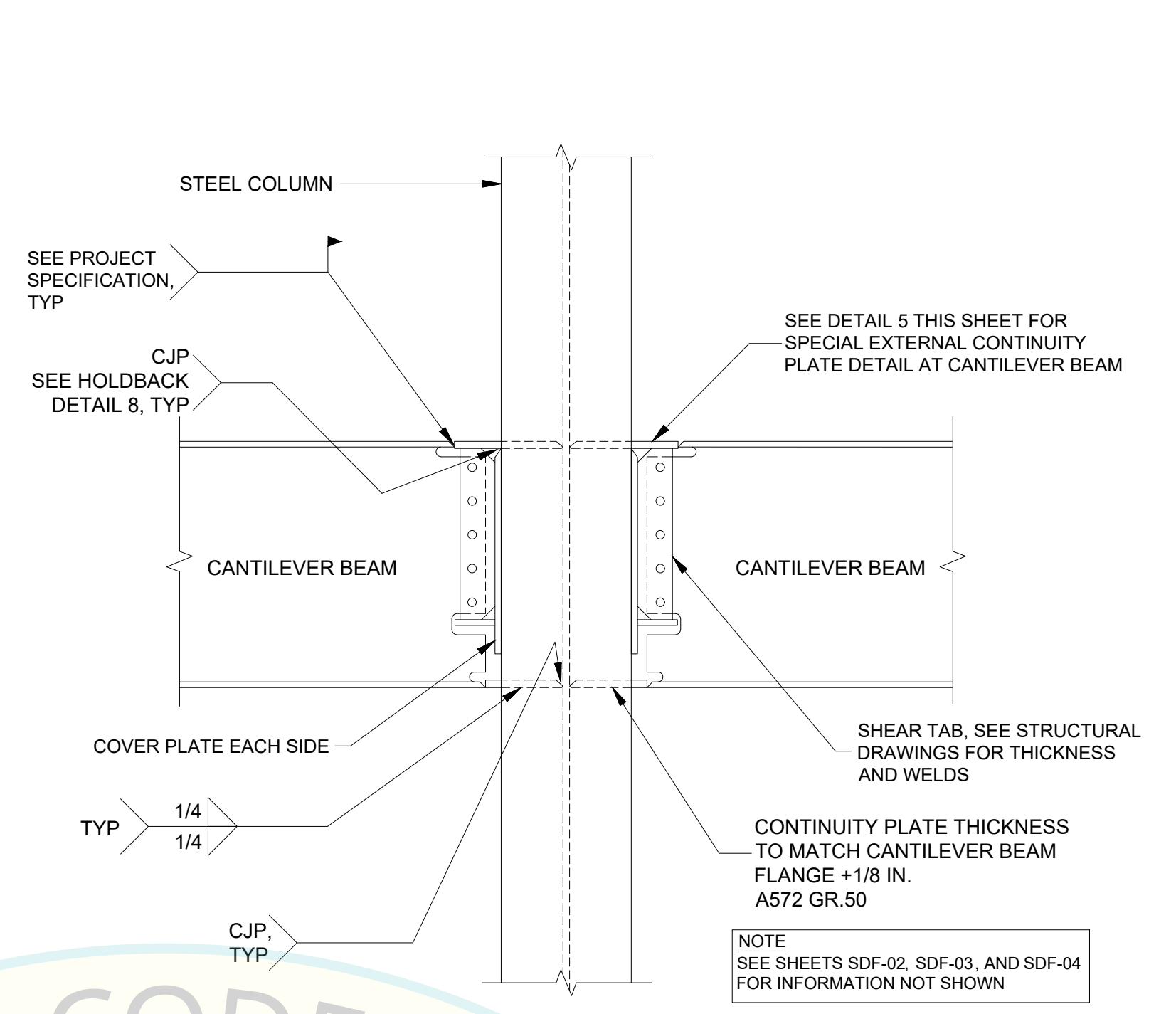
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 SHEET NUMBER: **SDF-04**



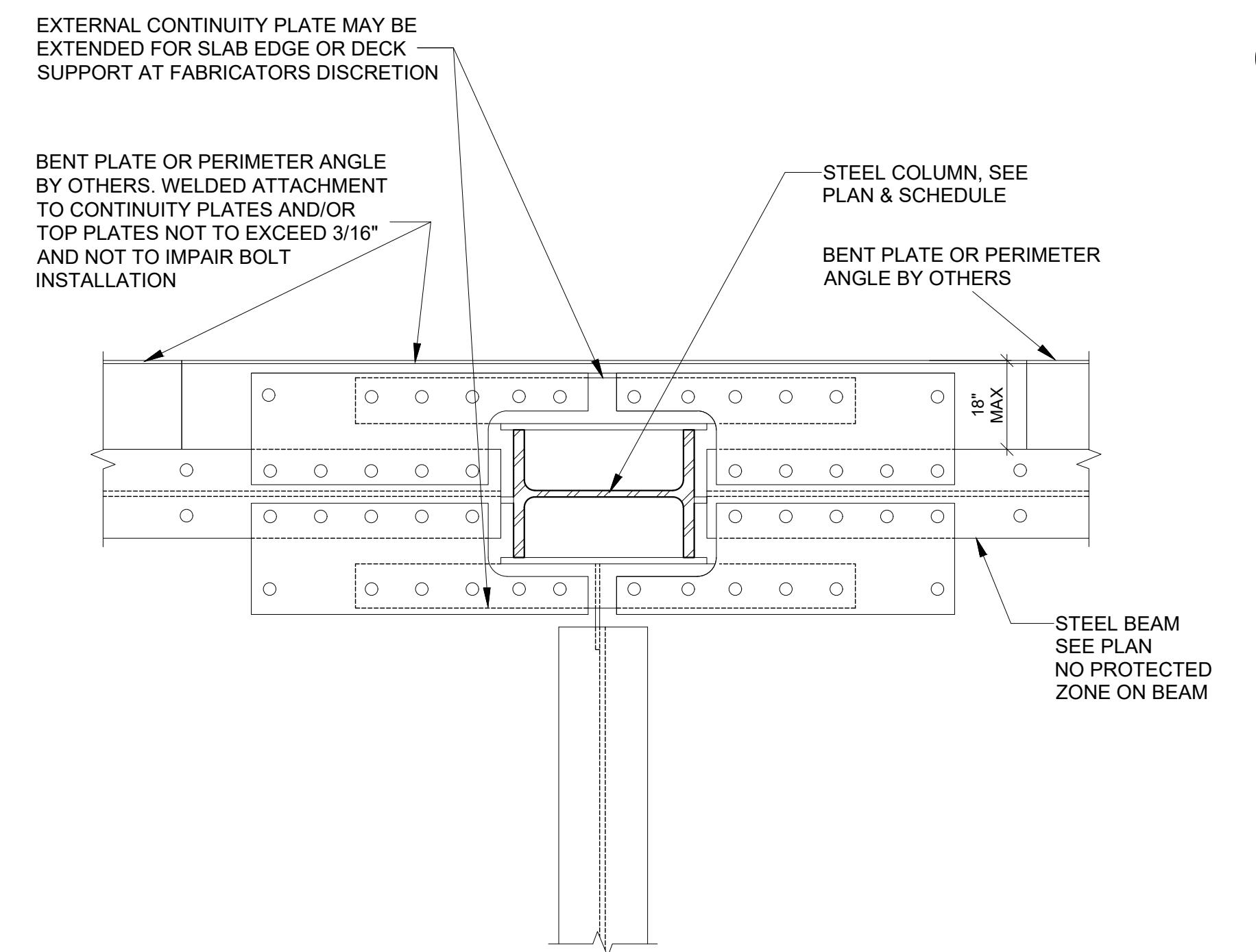
1 CANTILEVER BEAM - CANTILEVER DEPTH < MOMENT FRAME BEAM DEPTH
SCALE: NTS



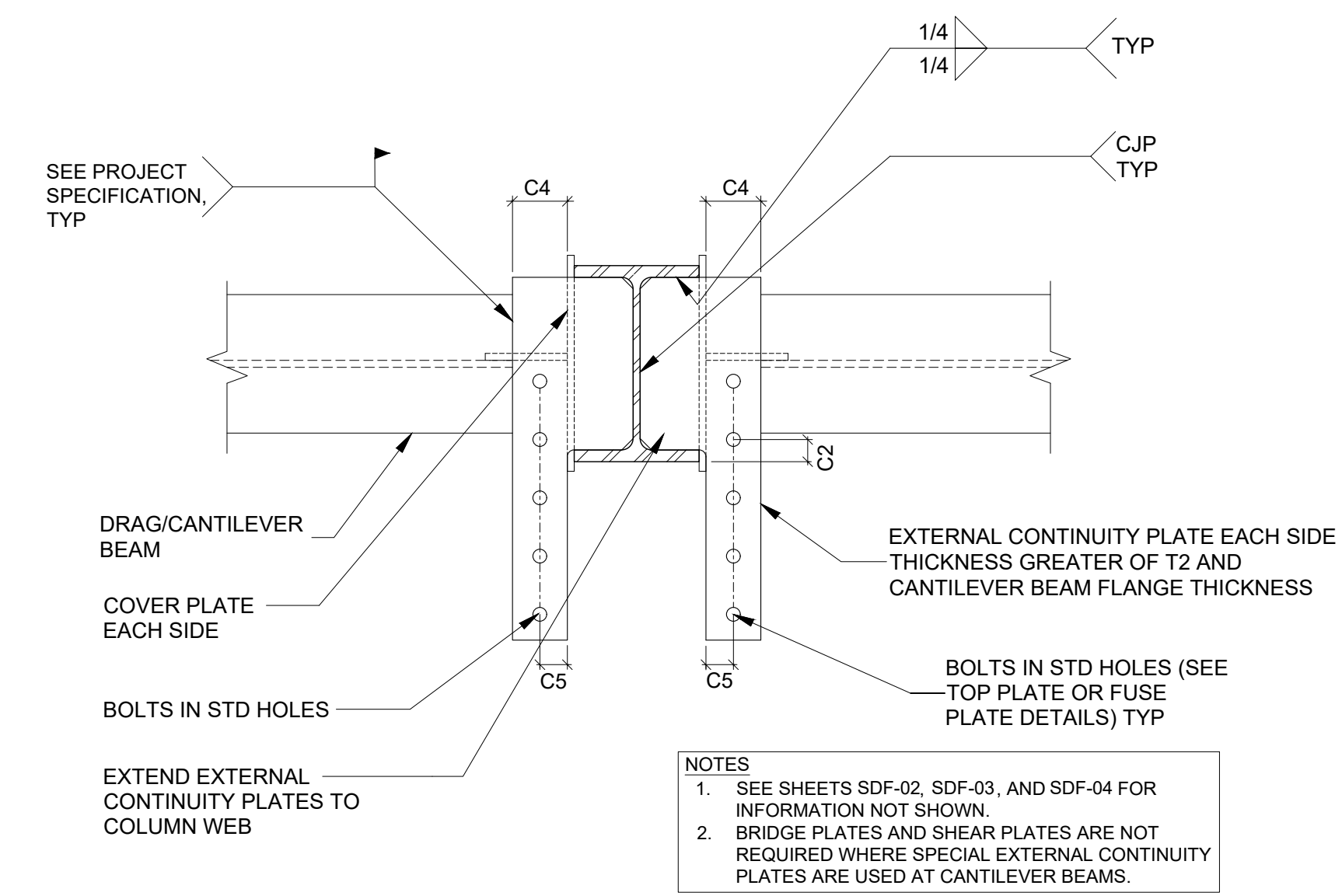
2 CANTILEVER BEAM - CANTILEVER DEPTH = MOMENT FRAME BEAM DEPTH
SCALE: NTS



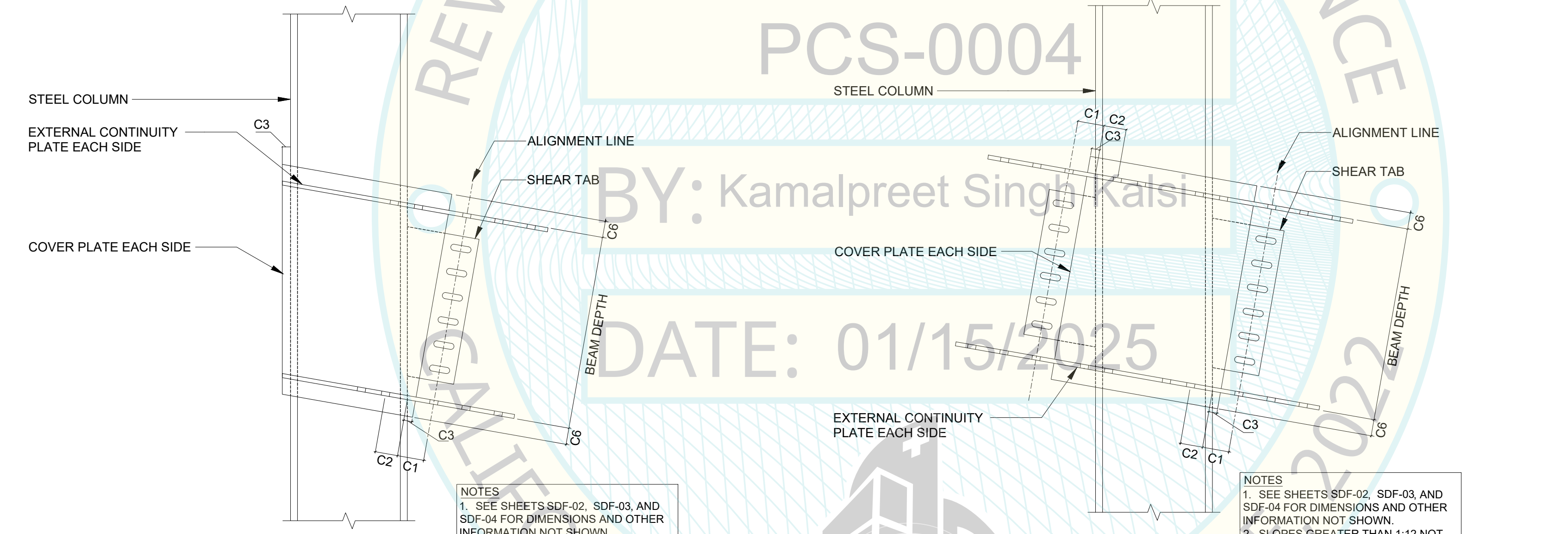
3 CANTILEVER BEAM - CANTILEVER DEPTH > MOMENT FRAME BEAM DEPTH
SCALE: NTS



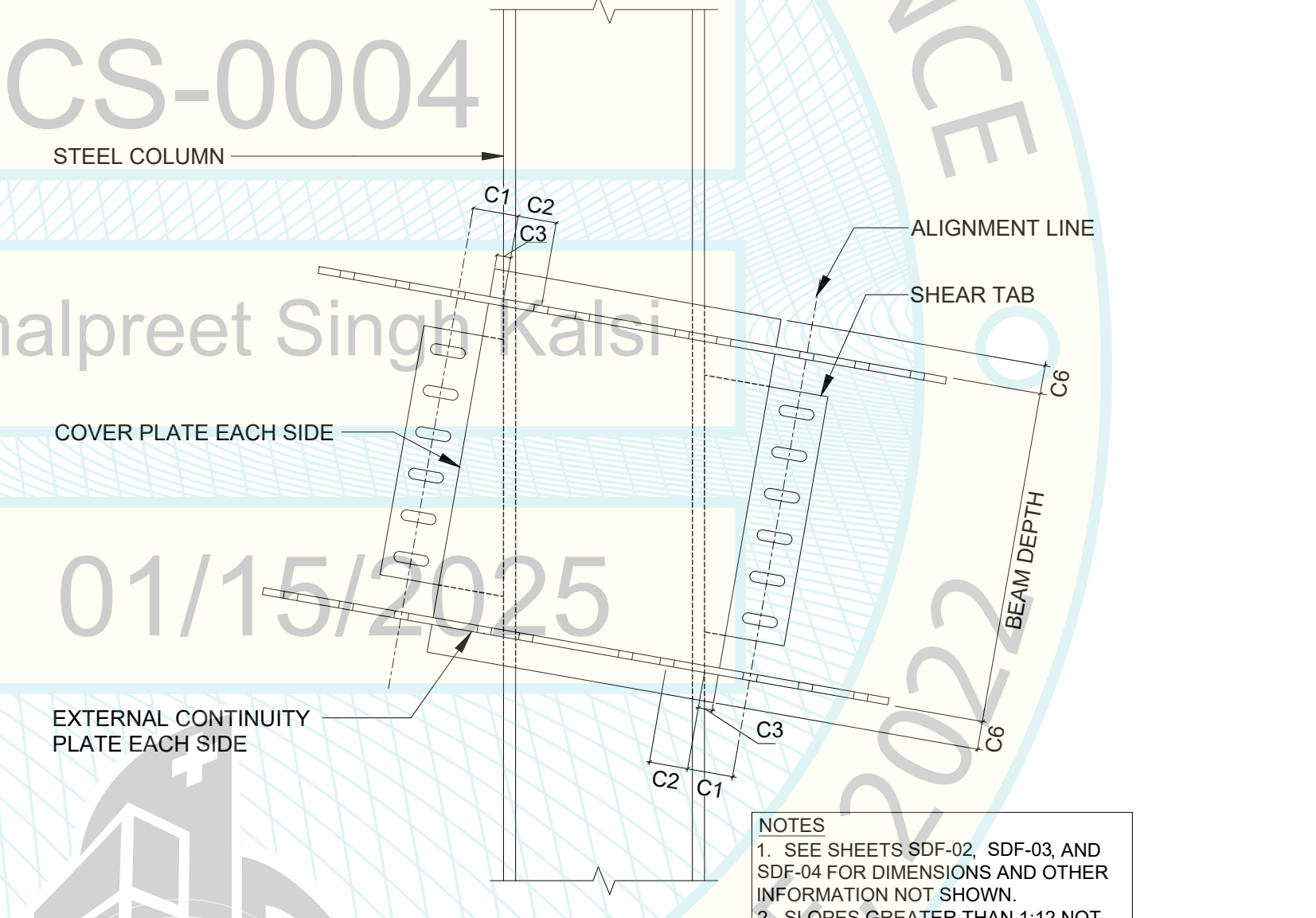
4 SLAB EDGE AND DECK SUPPORT
SCALE: NTS



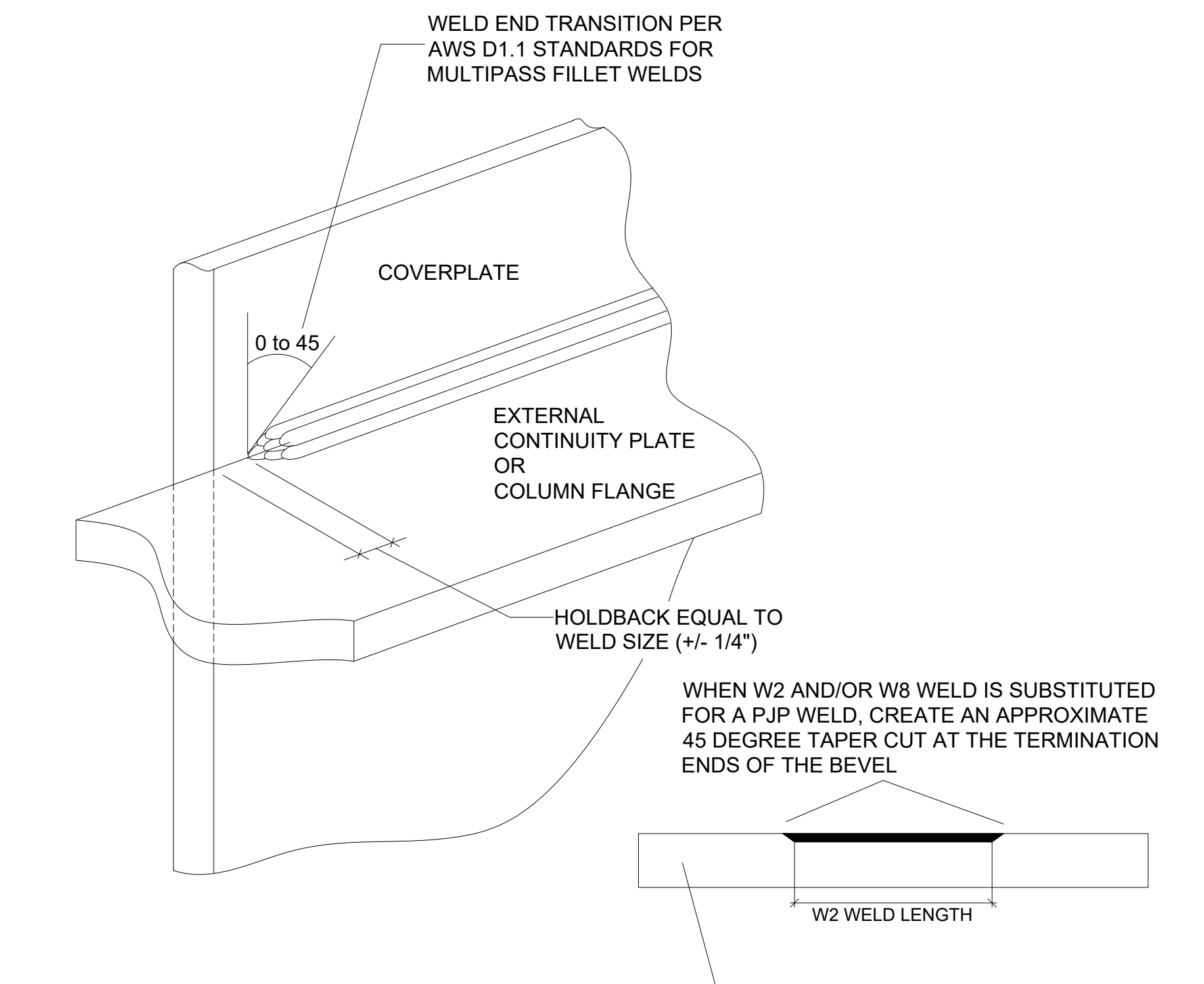
5 SPECIAL EXTERNAL CONTINUITY PLATE AT DRAG/CANTILEVER BEAM
SCALE: NTS



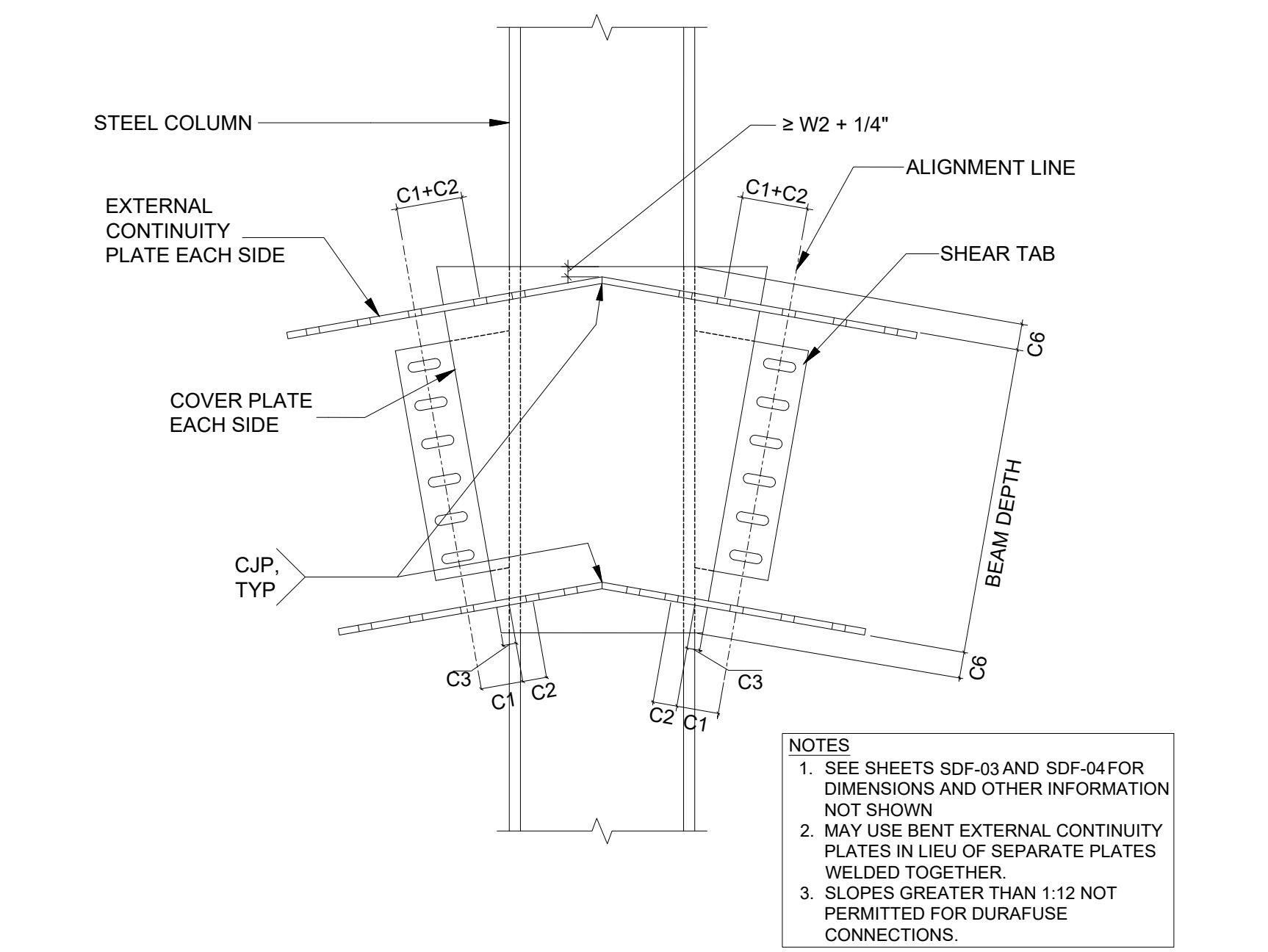
6 ONE SIDED SLOPED CONNECTION
SCALE: NTS



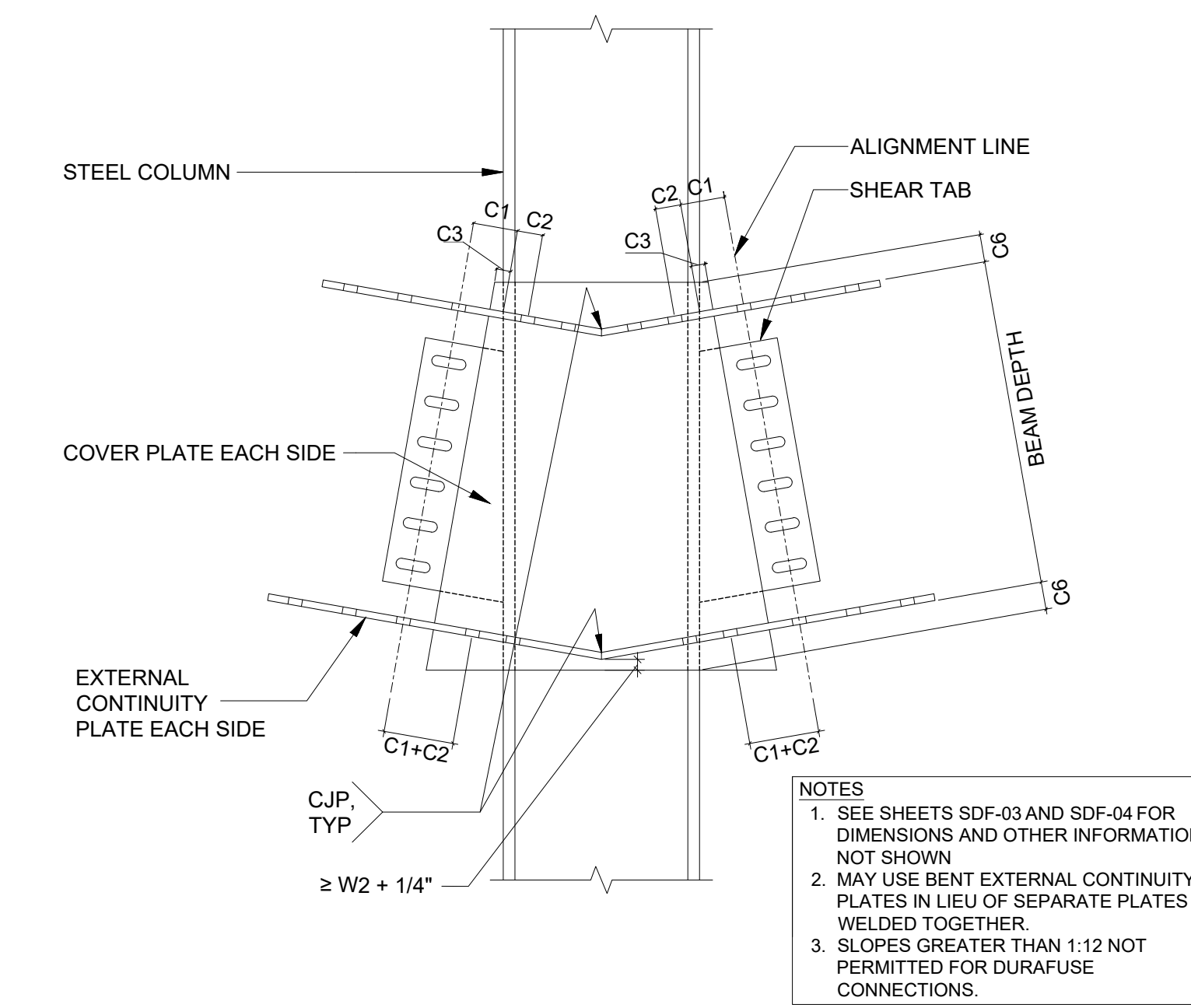
7 TWO SIDED SLOPED CONNECTION
SCALE: NTS



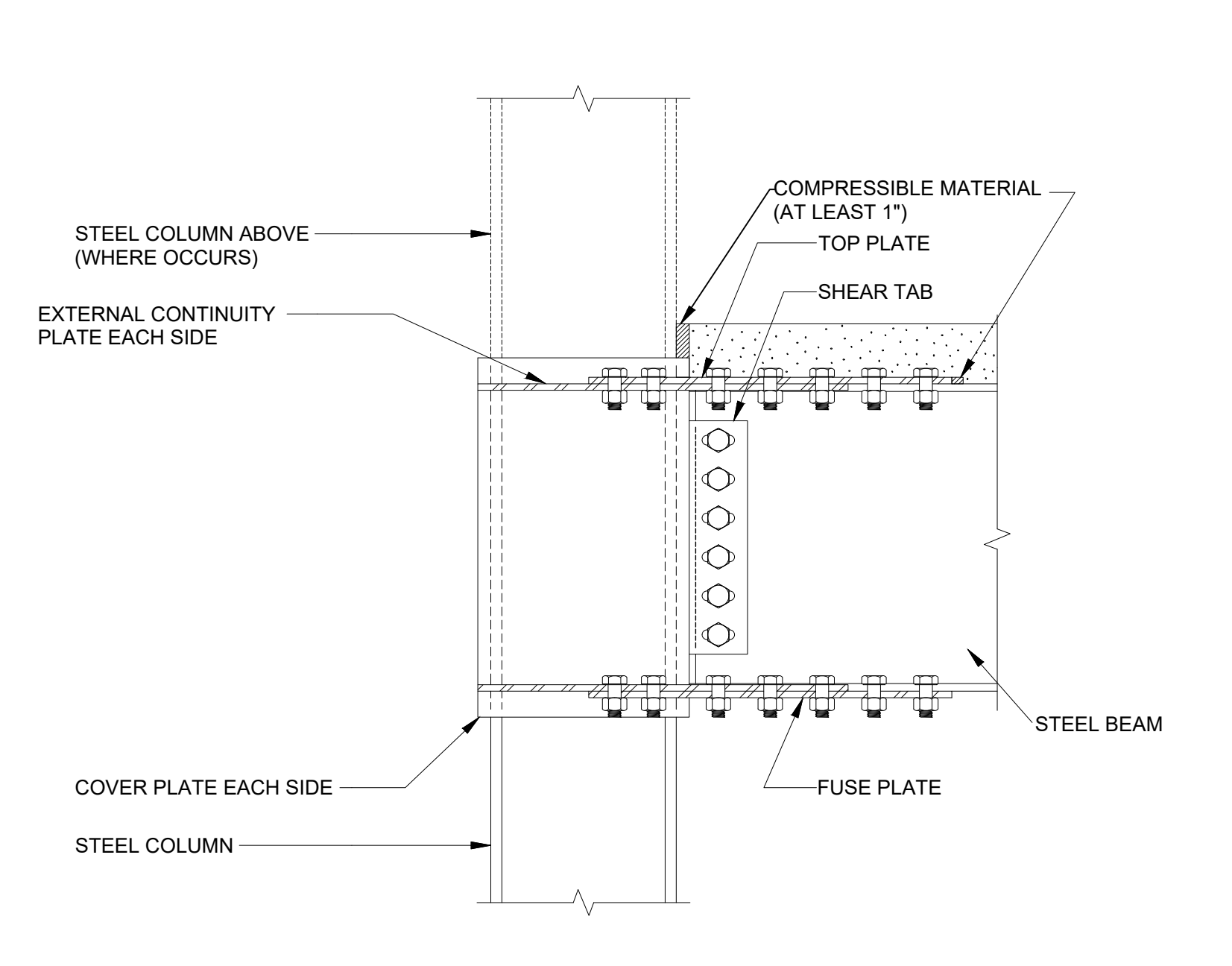
8 HOLDBACK DETAIL
SCALE: NTS



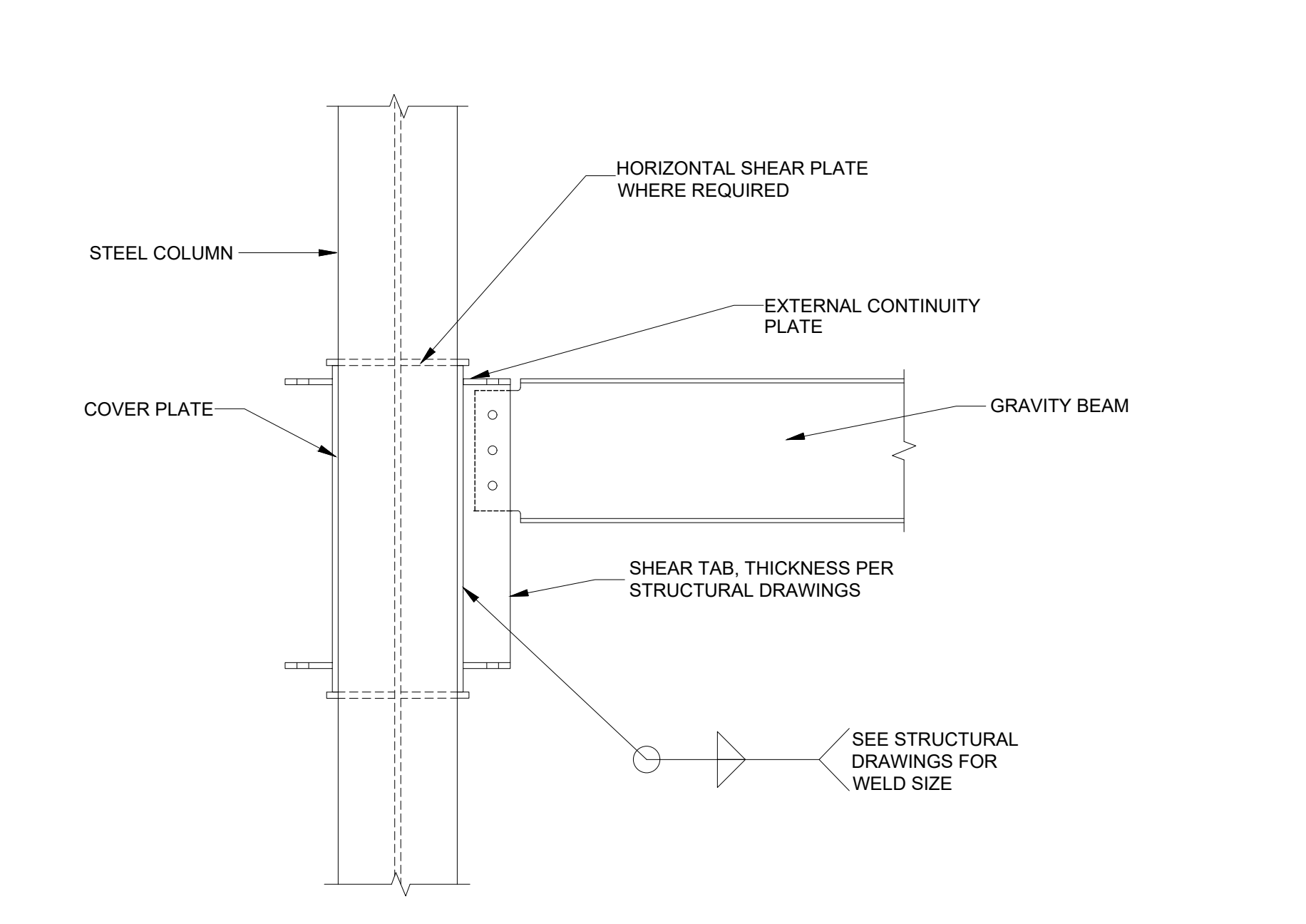
9 TWO SIDED DOUBLE SLOPED CONNECTION
SCALE: NTS



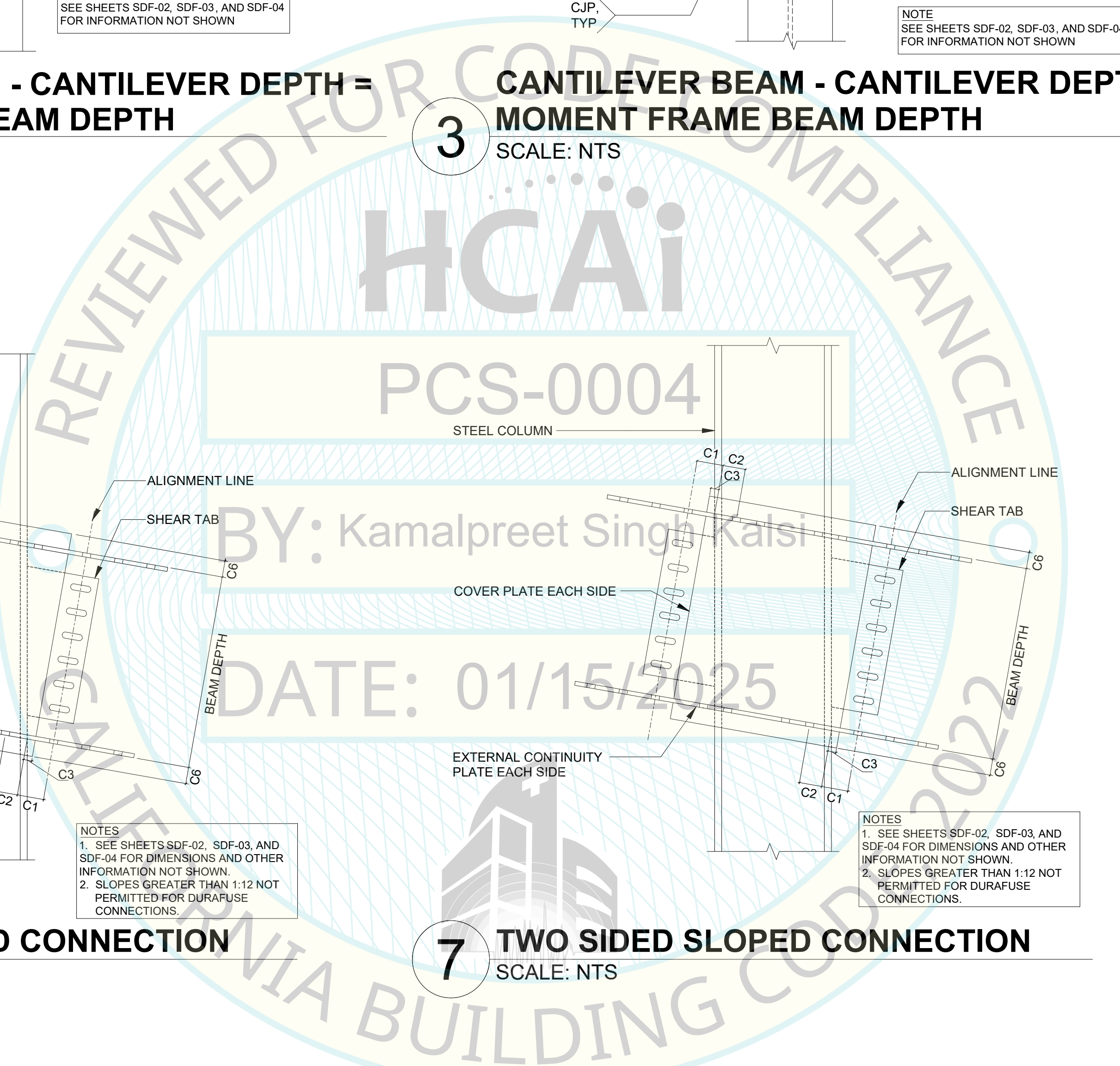
10 TWO SIDED DOUBLE SLOPED CONNECTION
SCALE: NTS



11 COMPRESSIBLE MATERIAL AT SLAB-TO-COLUMN INTERFACE
SCALE: NTS



12 GRAVITY BEAM TO DURAFUSE CONNECTION
SCALE: NTS



DESCRIPTION	DATE	NO.

ADDITIONAL NOTES:
HCAI

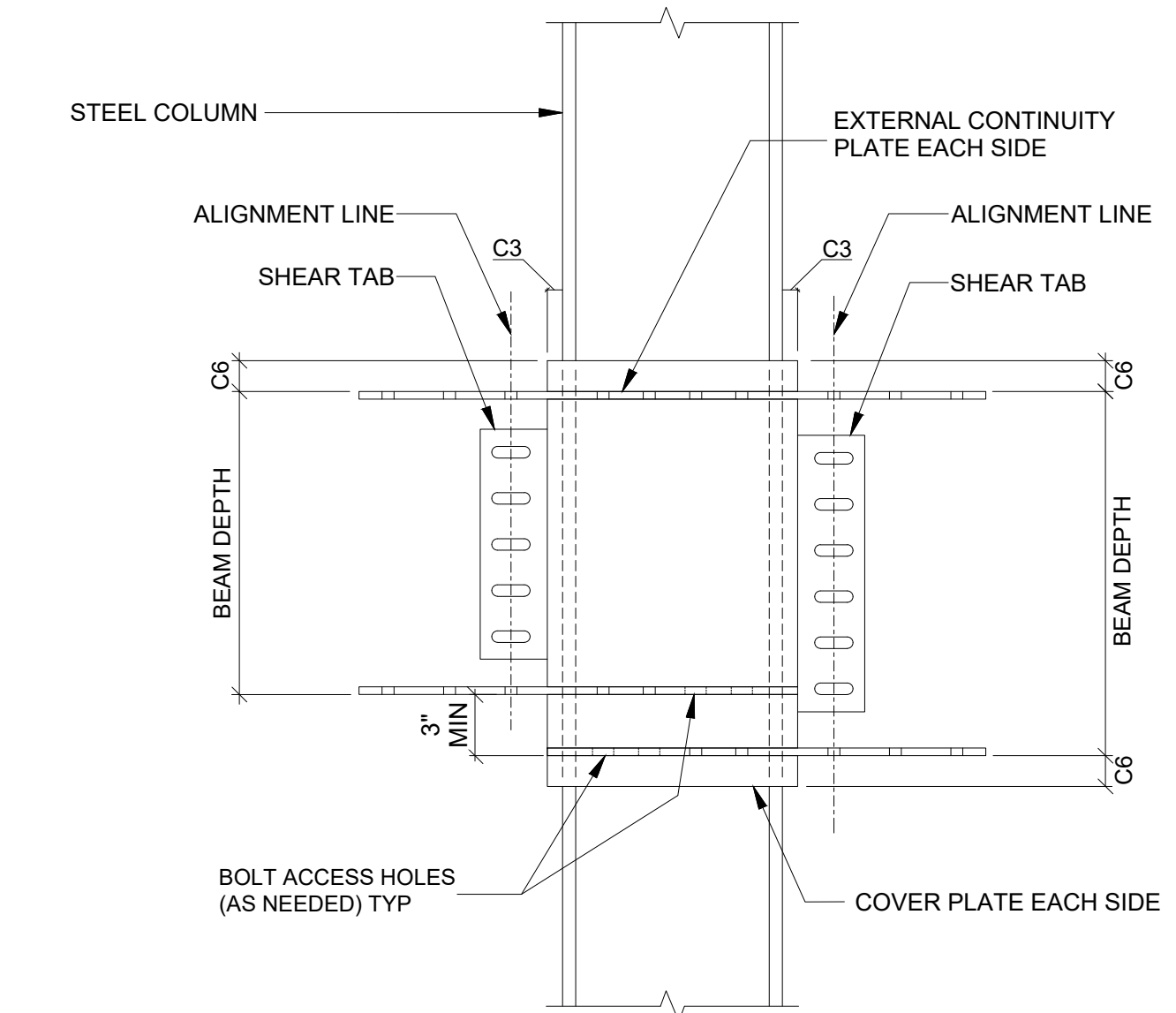
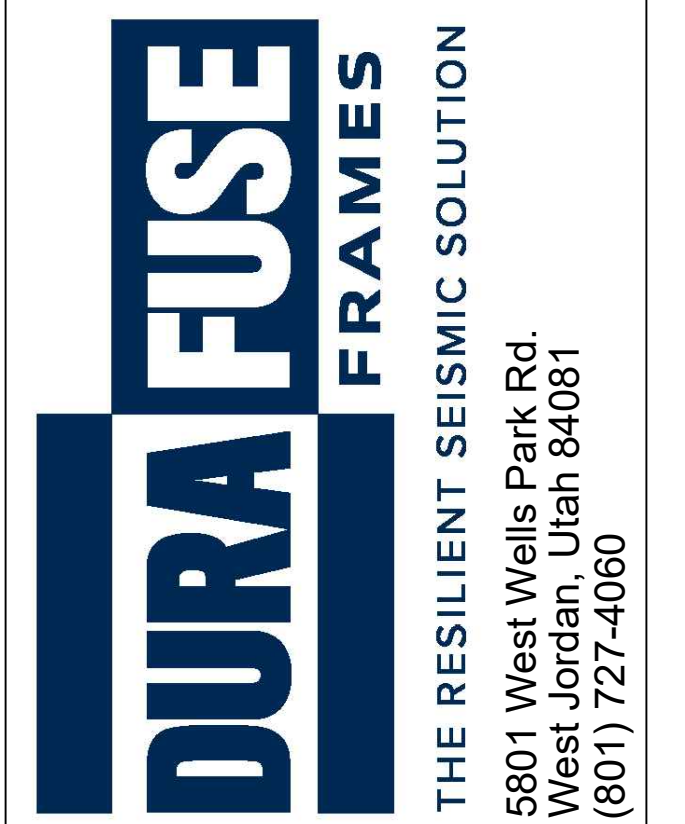
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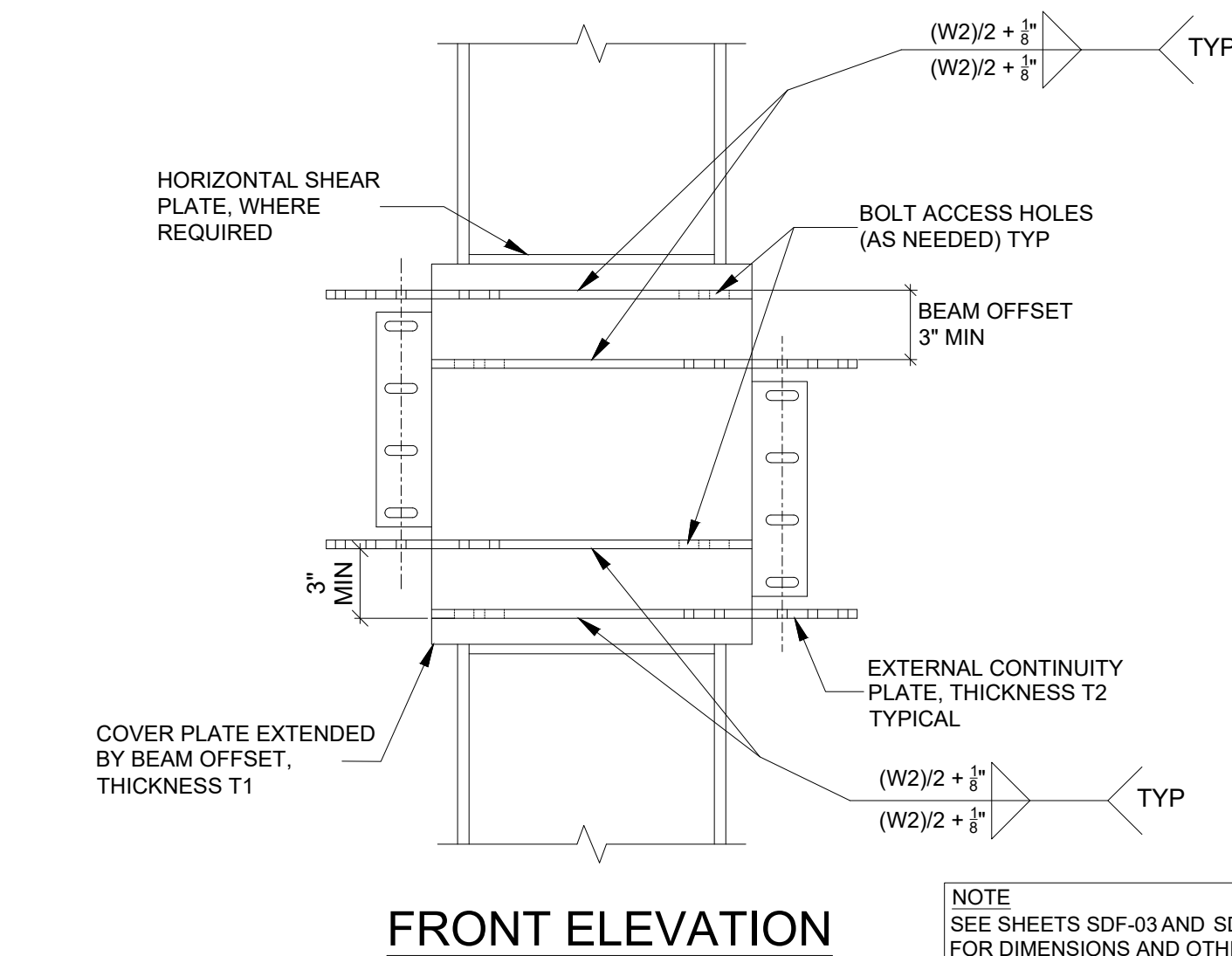
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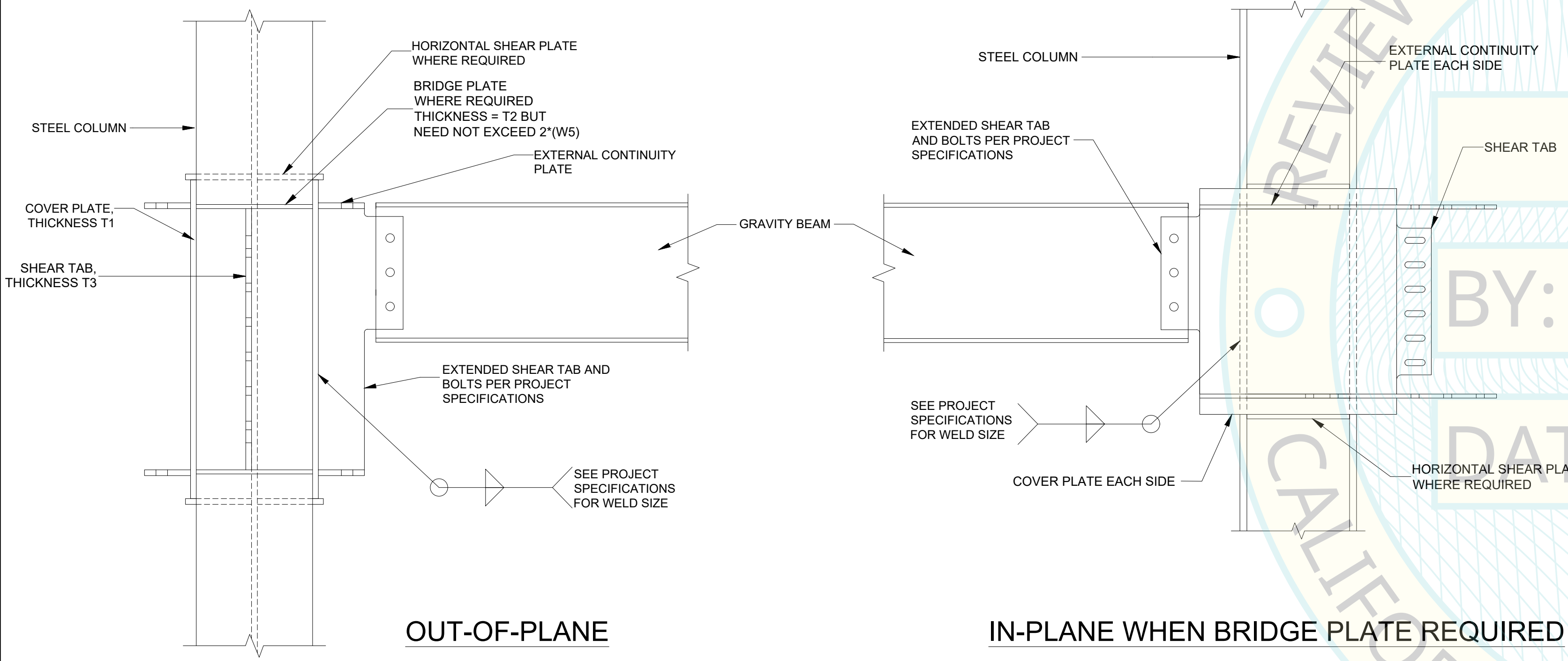
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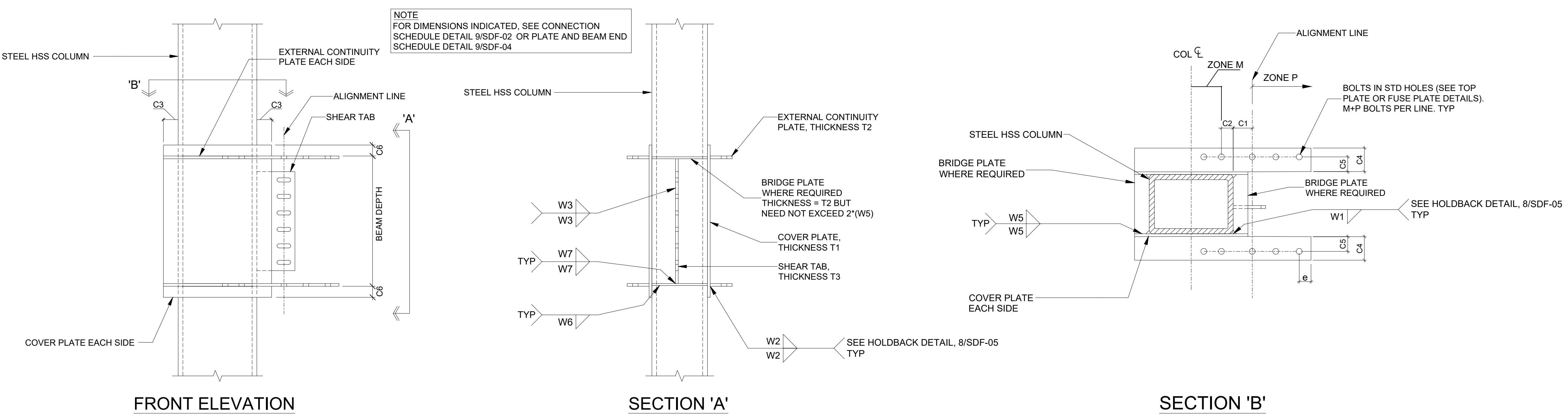
4 TWO SIDED CONNECTION - DIFFERENT BEAMS
SCALE: NTS



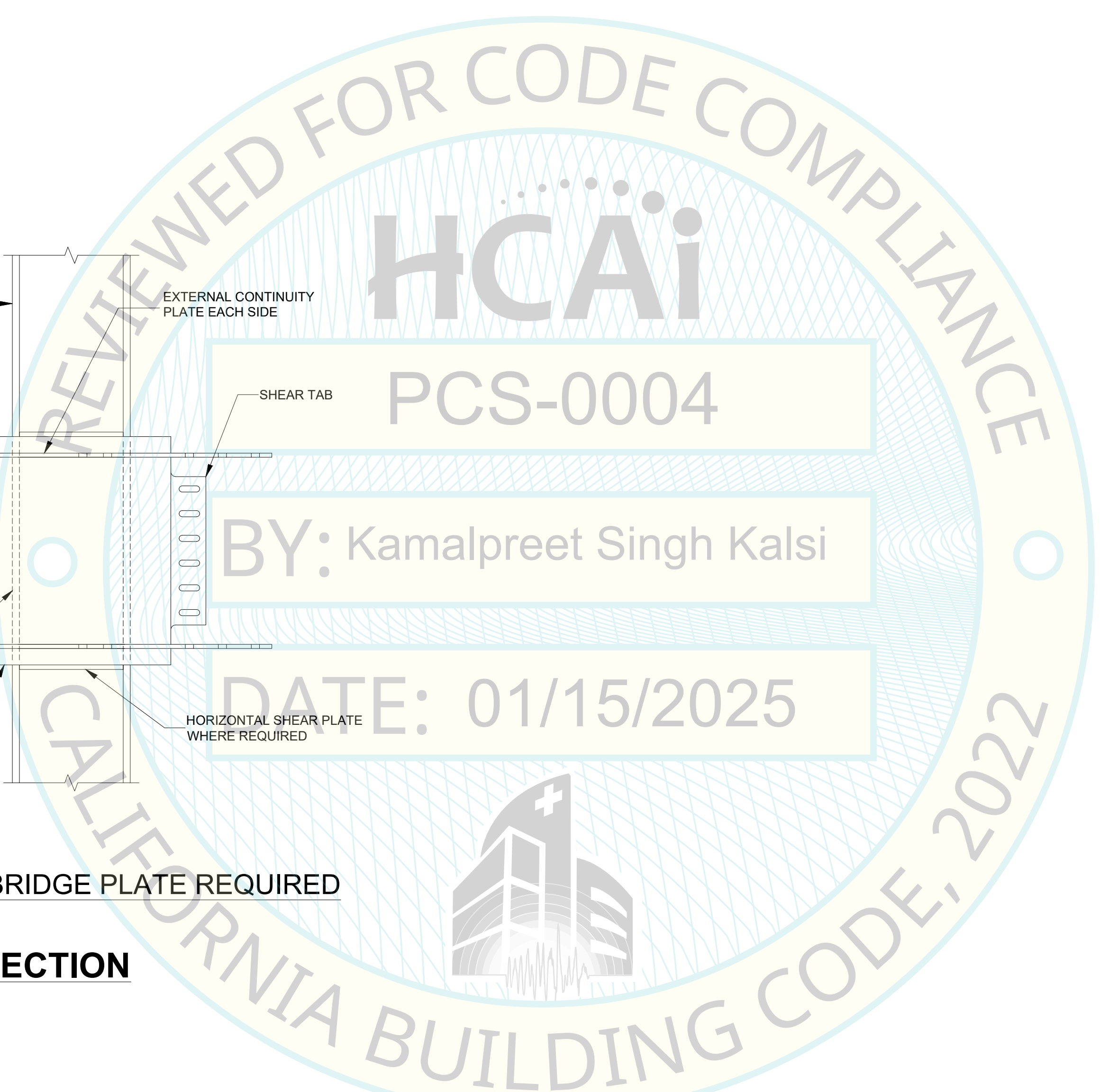
8 DROPPED BEAM CONNECTION - LARGE OFFSET
SCALE: NTS



5 EXTENDED SHEAR TAB FOR GRAVITY BEAM AT DURAFUSE CONNECTION
SCALE: NTS



9 ONE SIDED CONNECTION - HSS COLUMN ASSEMBLY
SCALE: NTS



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NOTES
 1. FOR DIMENSIONS INDICATED IN TYPICAL DETAILS THIS SHEET, SEE CONNECTION SCHEDULE DETAILS 9/SDF-02 AND 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-07 AND 9/SDF-04.
 2. WELDS W8 & W9 MAY BE SUBSTITUTED FOR AN EQUIVALENT FJP WELD AT FABRICATORS DISCRETION.

PROJECT:

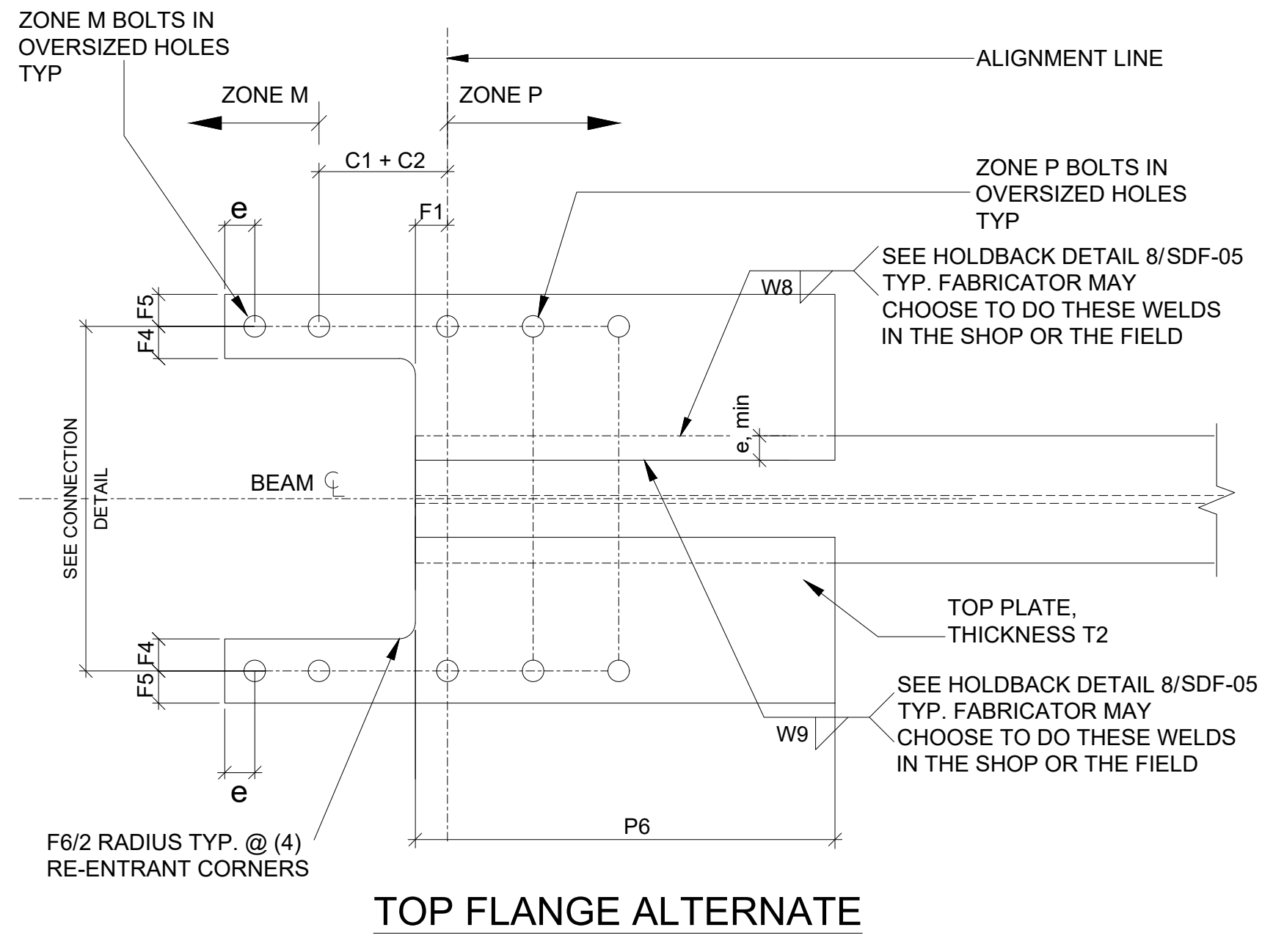
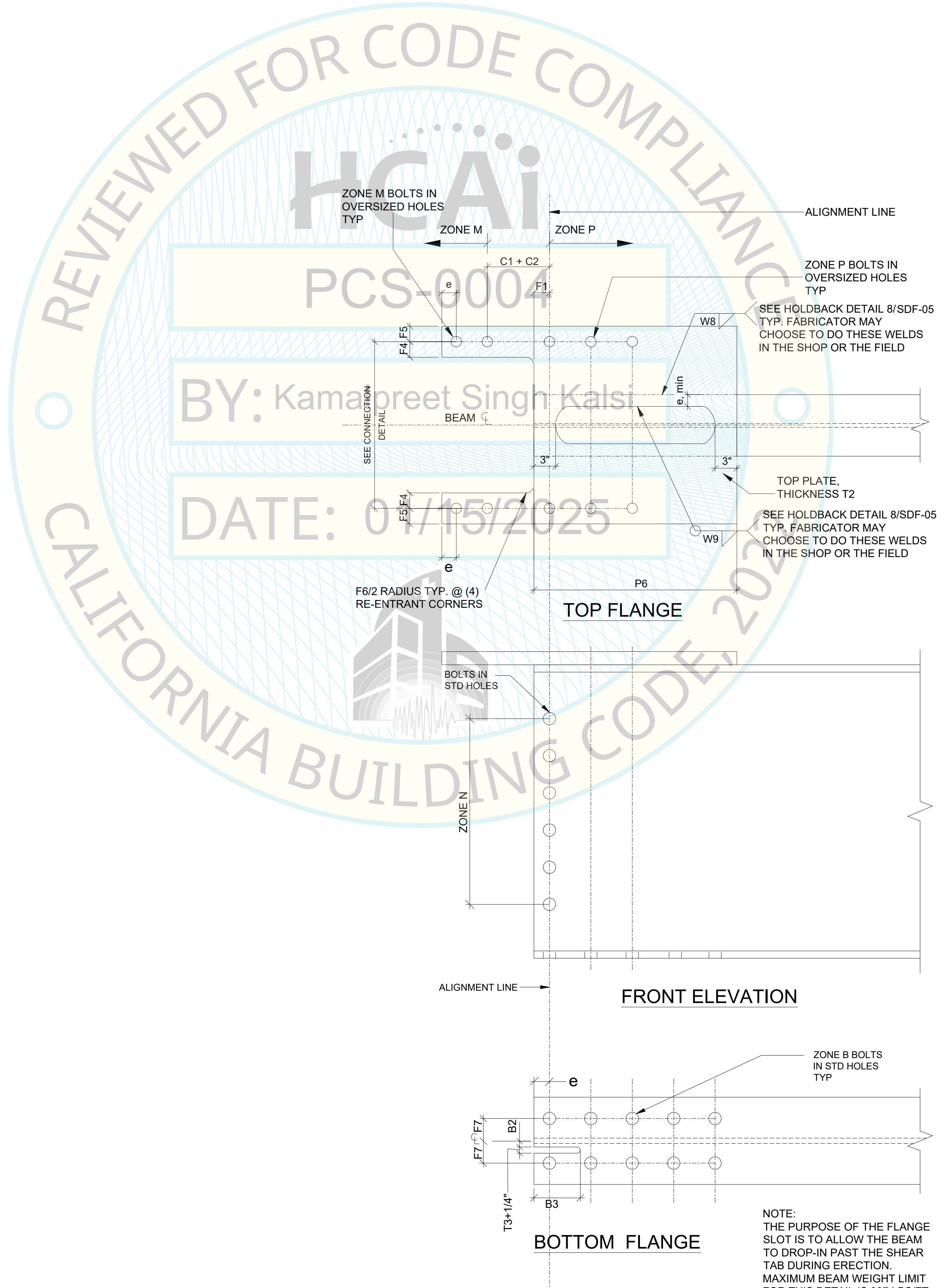
**HCAI
 CALCULATIONS**

XXXX, XX

DESCRIPTION	DATE	NO.

ADDITIONAL NOTES:
 HCAI

STAMP AND SIGNATURE:

ID	MEMBER SIZES		TOP PLATE PARAMETERS		
	COLUMN	BEAM	P6	W8	W9

NOTES:
 1. SEE CONNECTION SCHEDULES ON PREVIOUS SHEETS FOR DIMENSIONS NOT PROVIDED IN THIS TABLE.
 2. $e = 1-1/8"$ FOR $3/4"$ BOLTS, $1-1/4"$ FOR $7/8"$ BOLTS, $1-3/8"$ FOR $1"$ BOLTS, $1-5/8"$ FOR $1-1/8"$ BOLTS, OR $1-3/4"$ FOR $1-1/4"$ BOLTS.
 3. IF B2 AND B3 EQUAL ZERO THEN THERE IS NO SLOT IN THE BEAM FLANGE, AND THE BEAM CAN SWING AROUND THE SHEAR TAB DURING ERECTION.

9 WELDED TOP PLATE AND BEAM END SCHEDULE
 SCALE: NTS

11 WELDED TOP PLATE & BEAM END
 SCALE: NTS

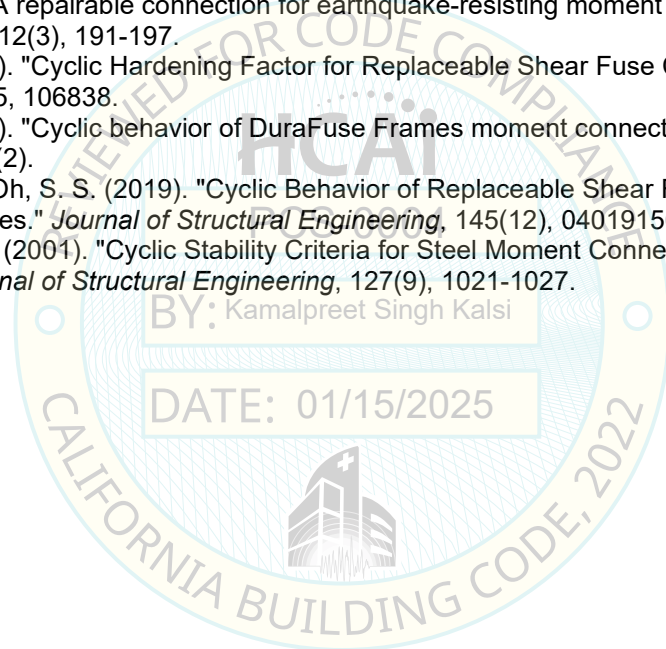
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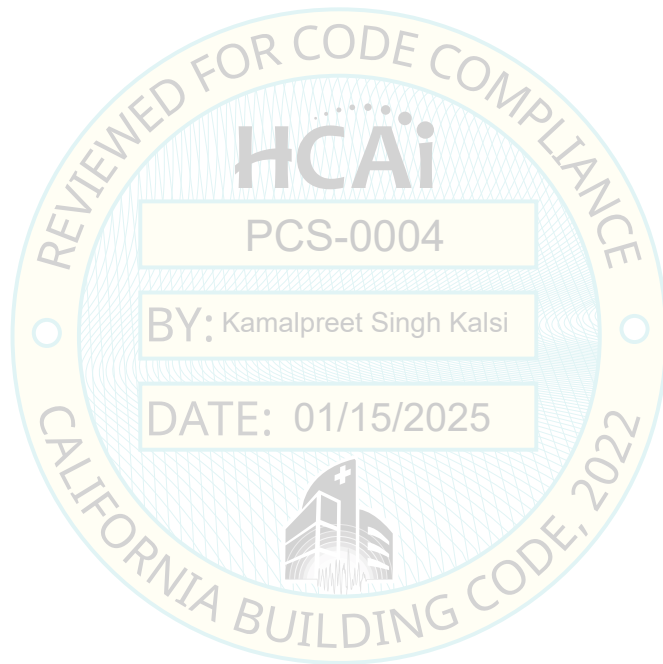
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9 Appendix A – Example Calculations



CALCULATION PACKAGE

HCAI CALCULATIONS

Software Version 1.8.81



1/15/2025

XXXXXX-00

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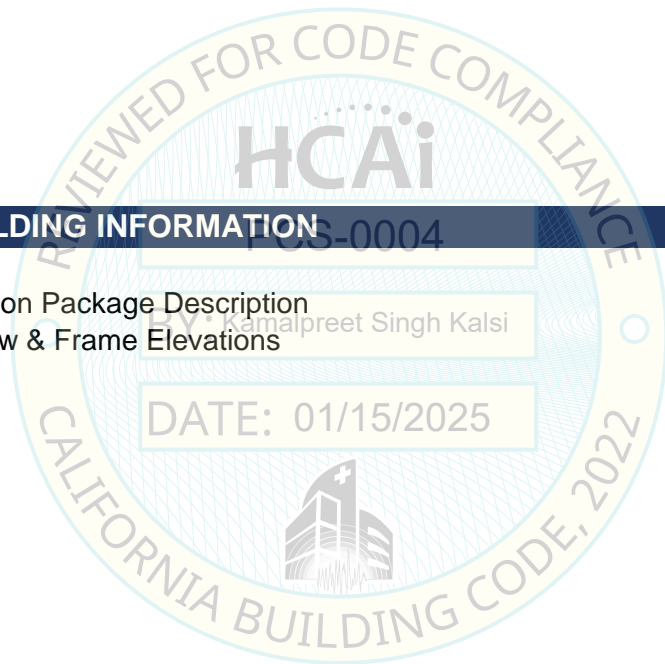
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1 GENERAL BUILDING INFORMATION

Calculation Package Description
Plan View & Frame Elevations

1.1
1.2



DURAFUSE FRAMES CONNECTION CALCULATION PACKAGE

HCAI CALCULATIONS

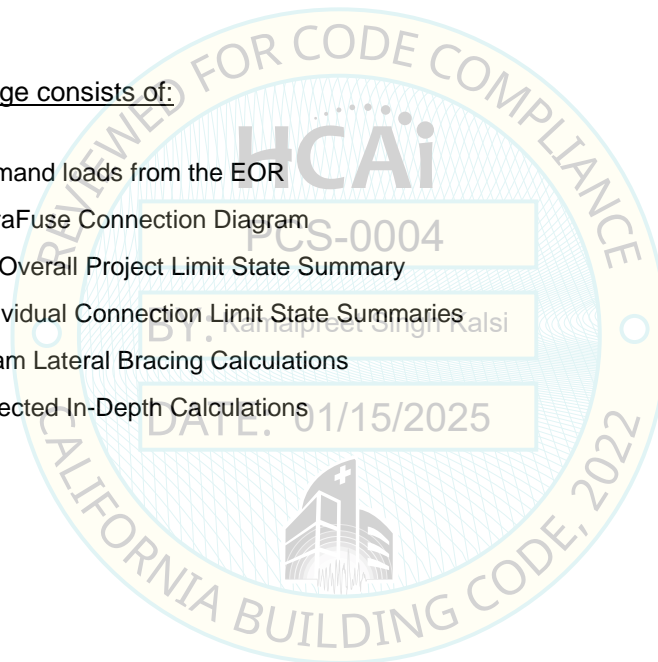
Software Version 1.8.81

Description

DuraFuse Frames has designed the moment connections for the above listed project. The provided calculations demonstrate satisfaction of the required limit states for each connection. Calculation results are first summarized for all connections, then detailed calculations are presented afterwards for individual connections. DuraFuse Frames is only responsible for the DuraFuse moment connections, and not for the moment frame members, or other moment connections used in the building.

This Calculation Package consists of:

- Demand loads from the EOR
- DuraFuse Connection Diagram
- An Overall Project Limit State Summary
- Individual Connection Limit State Summaries
- Beam Lateral Bracing Calculations
- Selected In-Depth Calculations



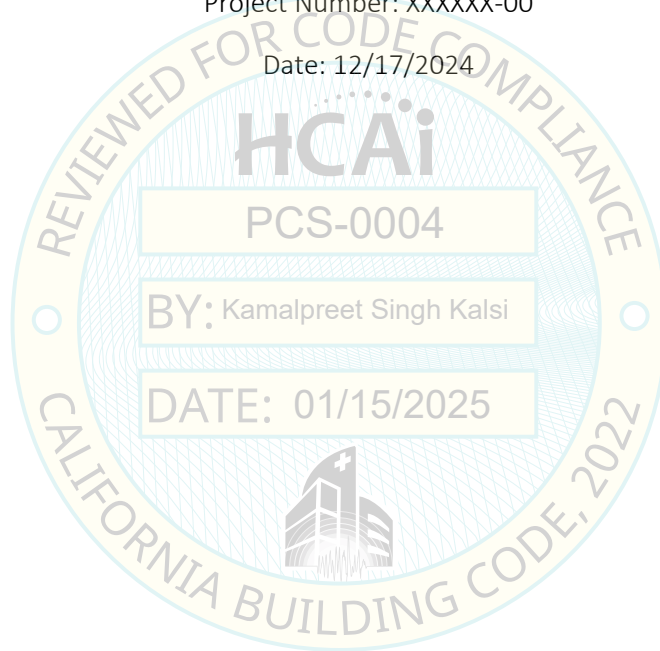


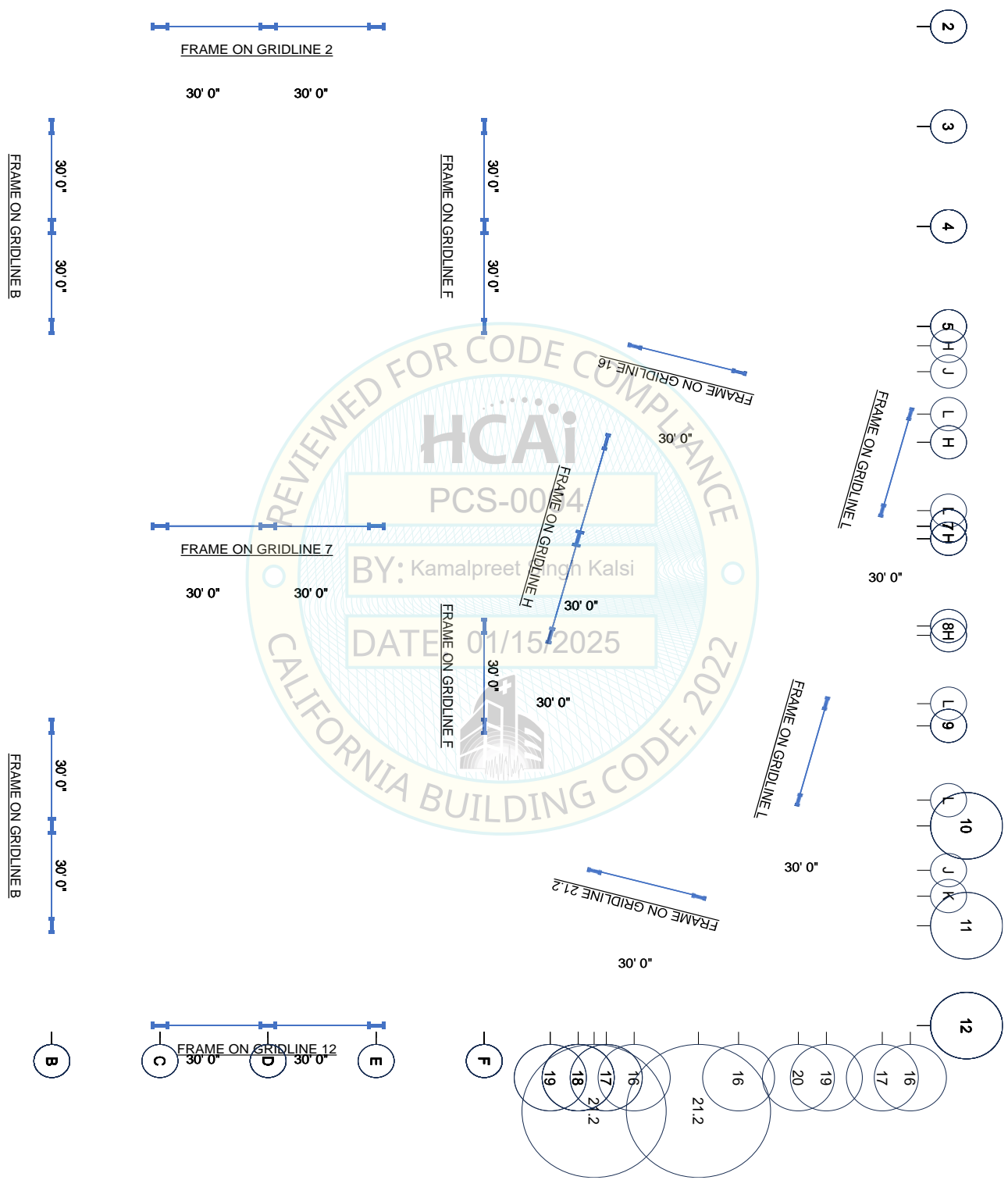
DuraFuse Frames Elevations

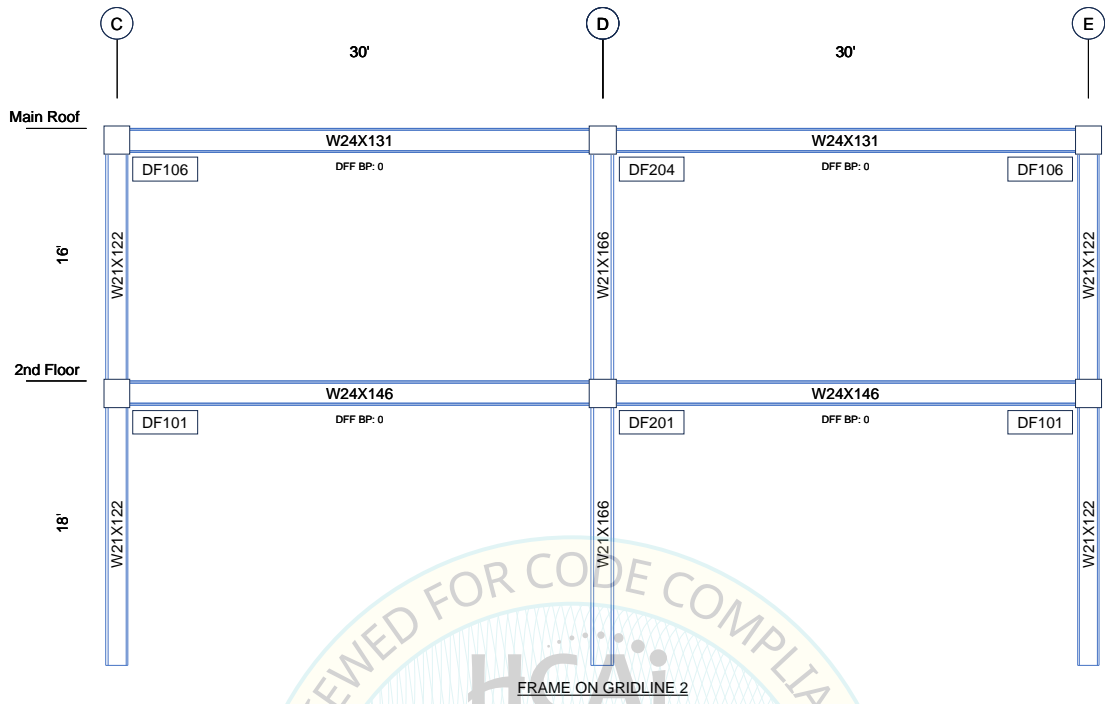
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Project Number: XXXXXX-00

Date: 12/17/2024

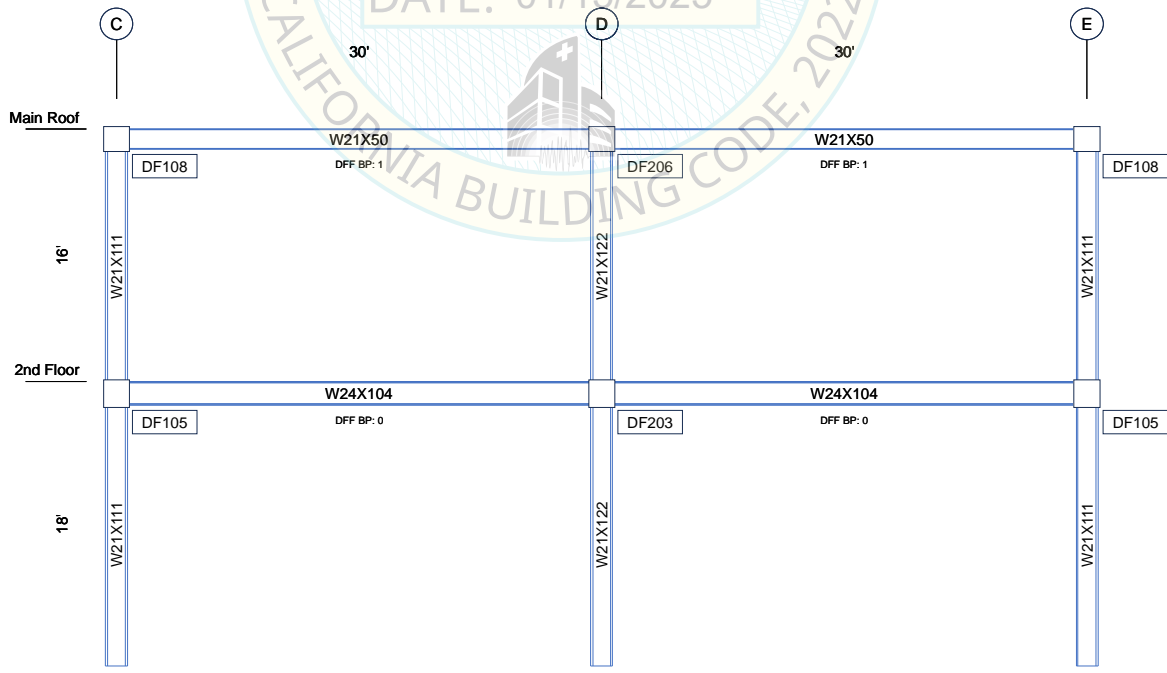
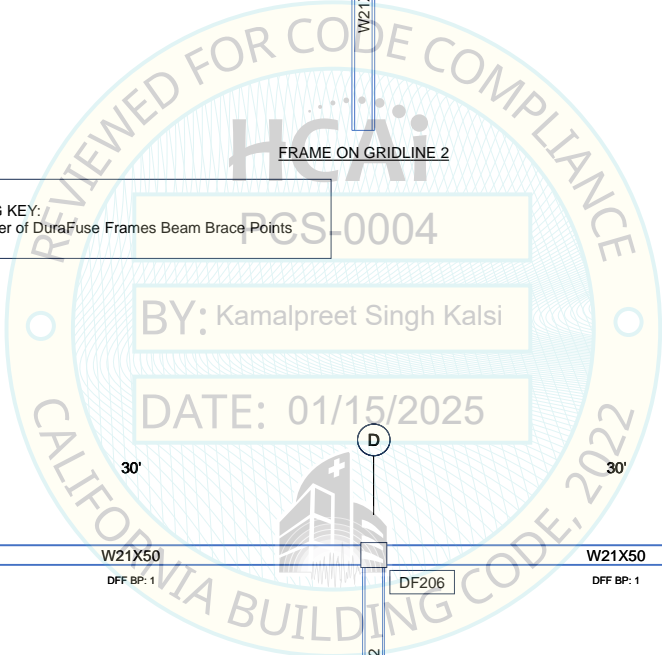




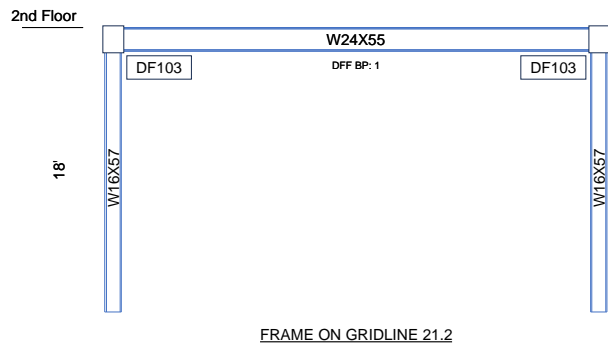
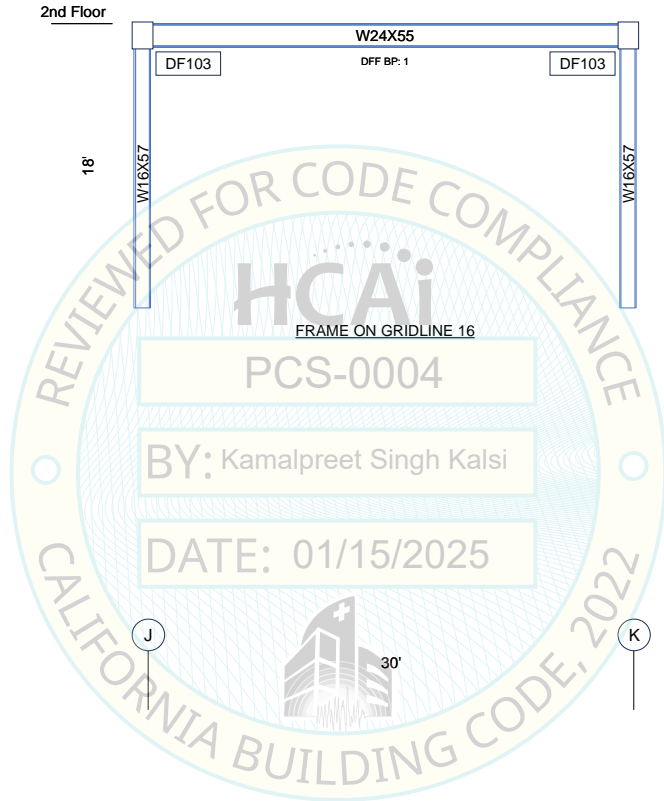


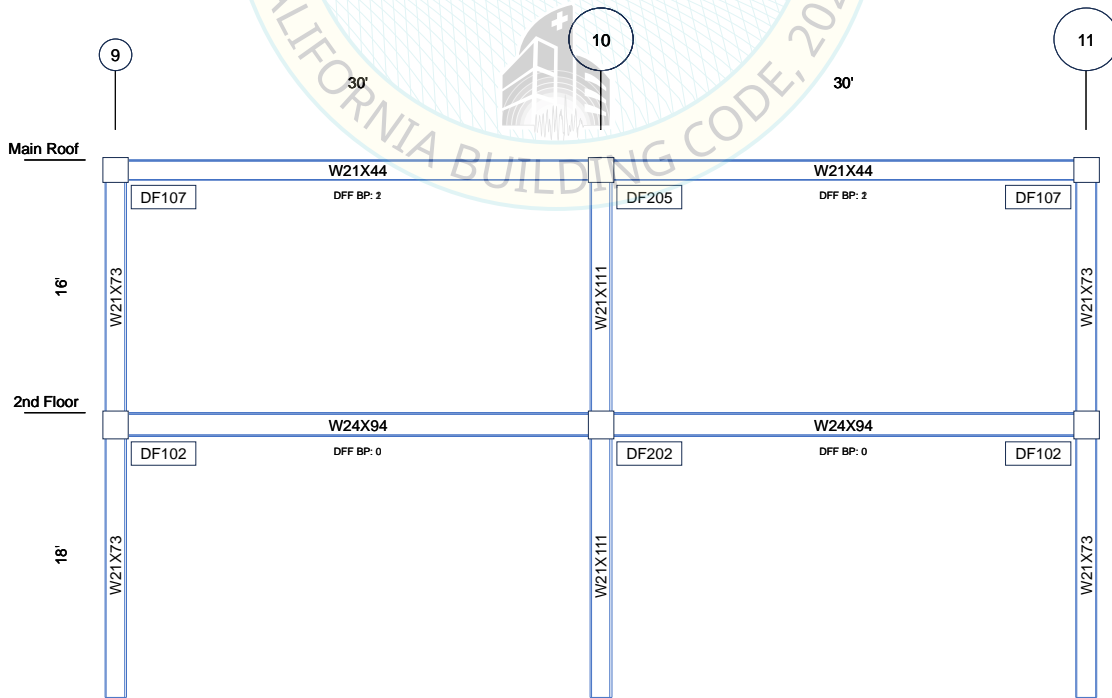
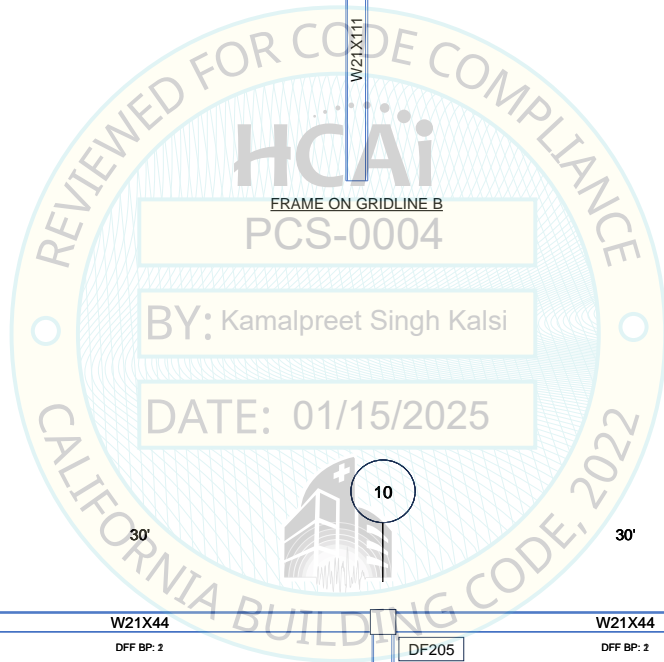
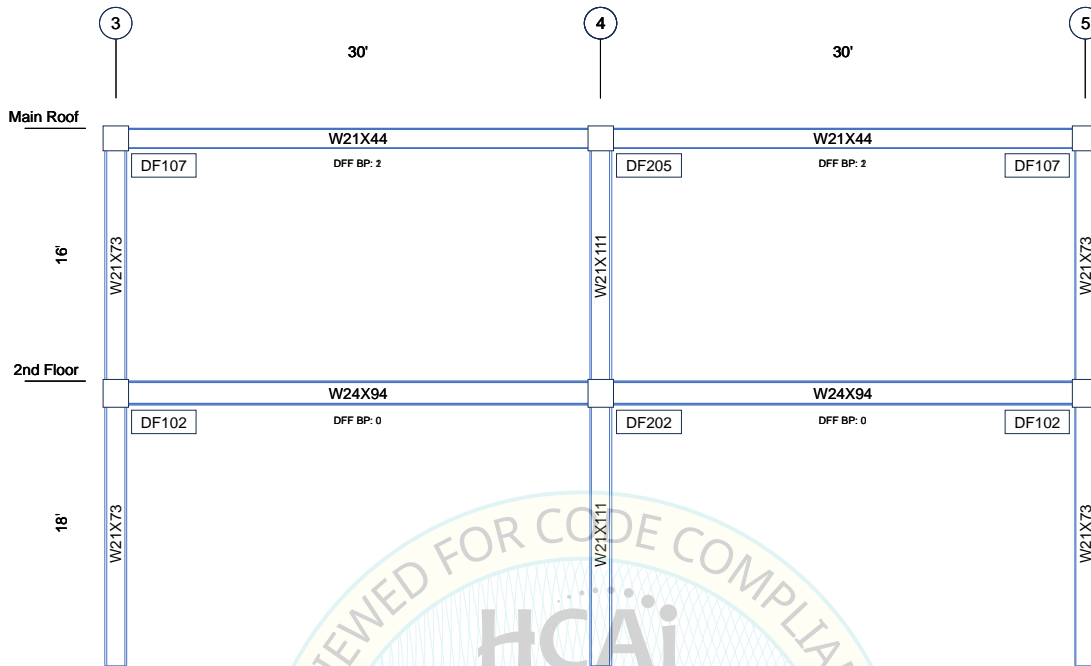
FRAME ON GRIDLINE 2

BEAM BRACING KEY:
 DFF BP = Number of DuraFuse Frames Beam Brace Points

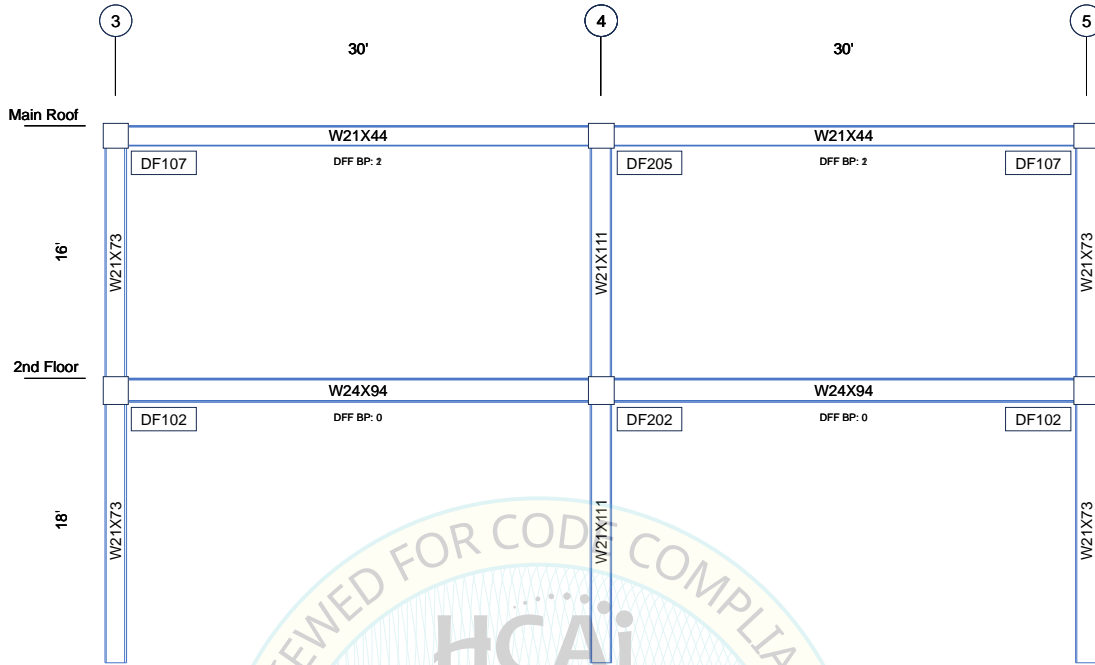


FRAME ON GRIDLINE 12





FRAME ON GRIDLINE B

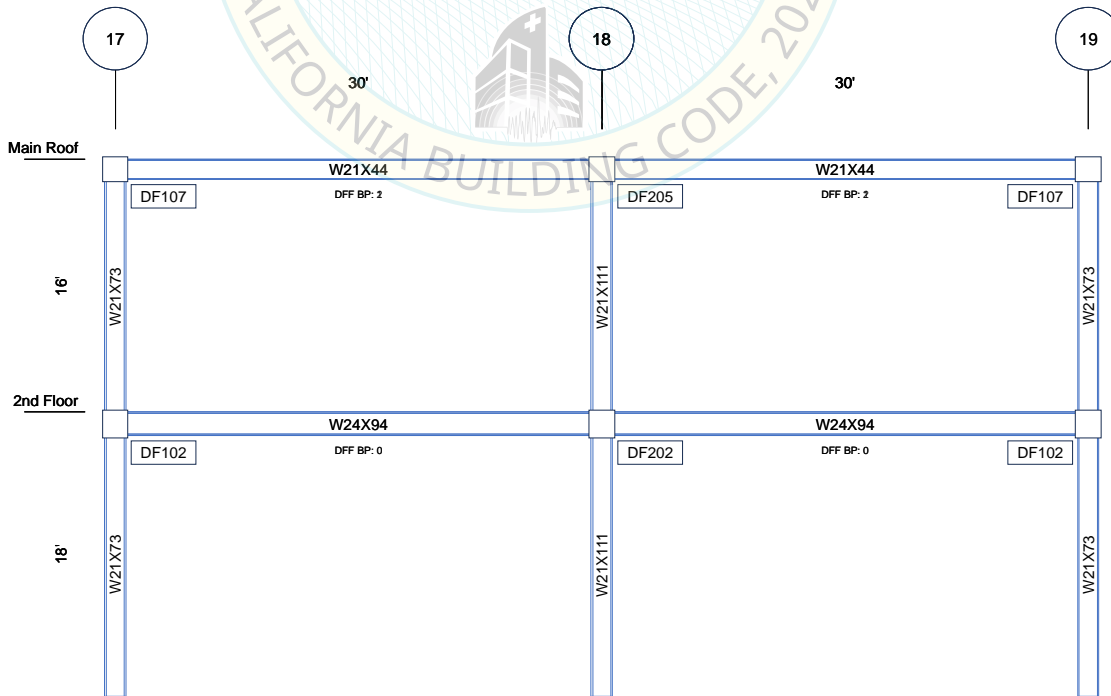


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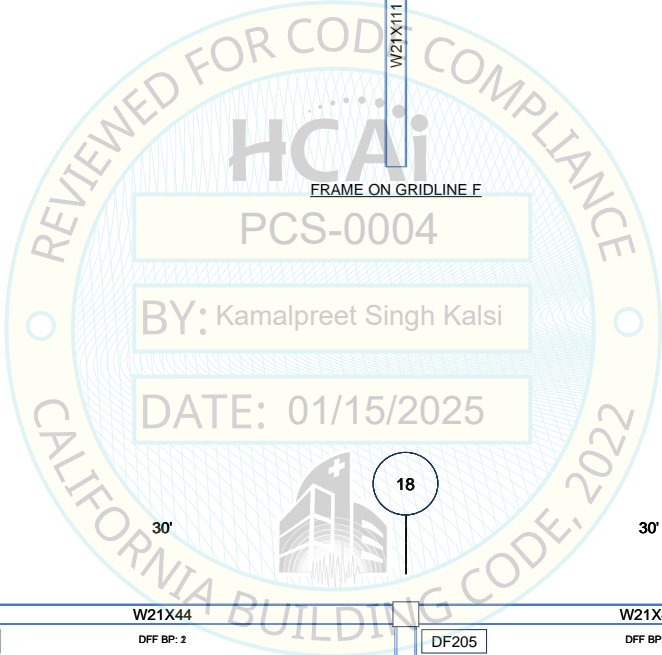
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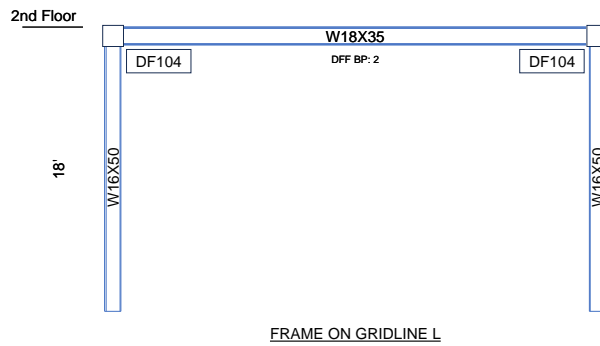
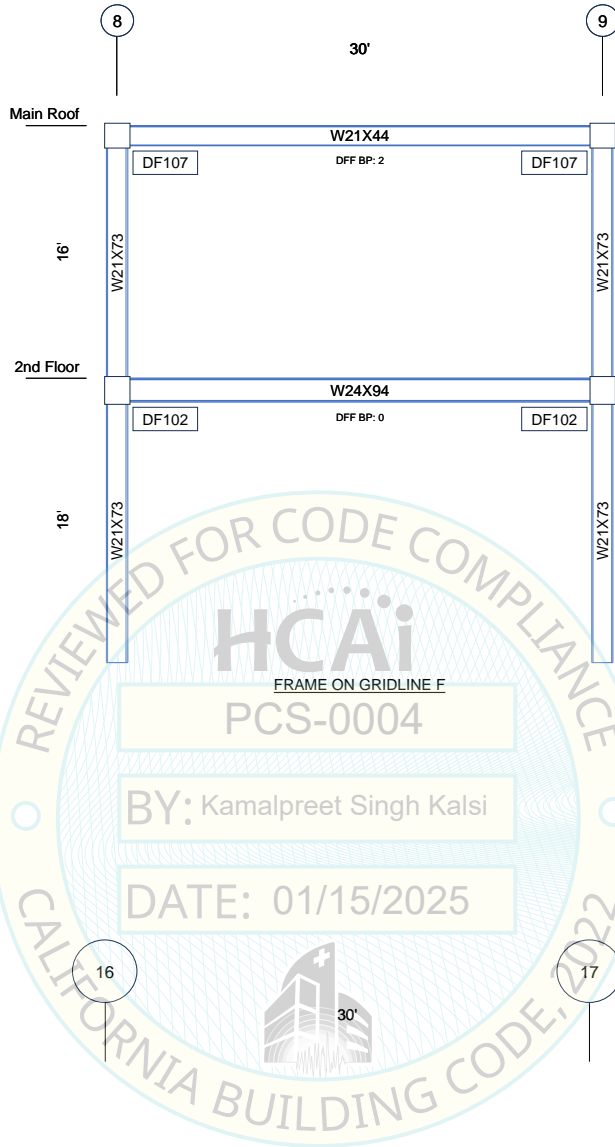
BY: Kamalpreet Singh Kalsi

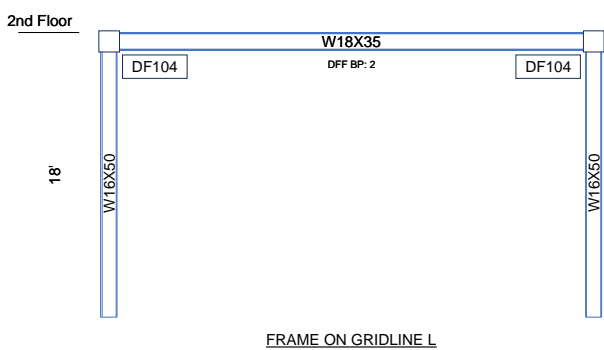
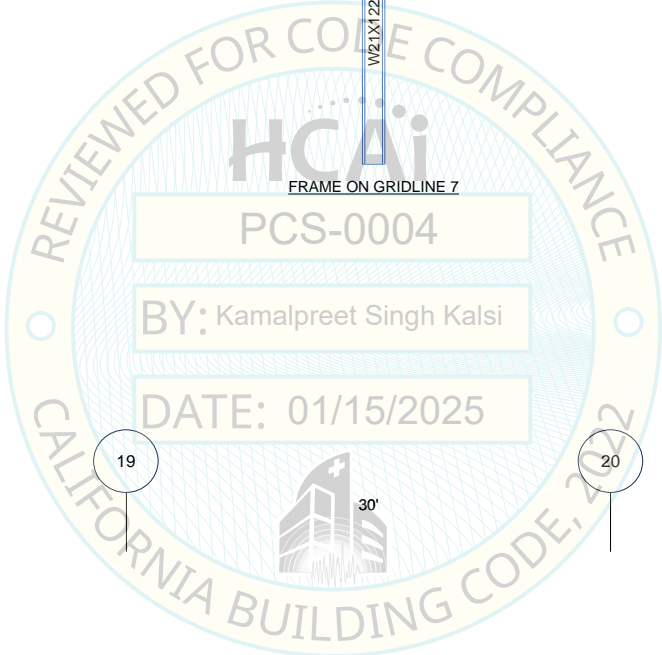
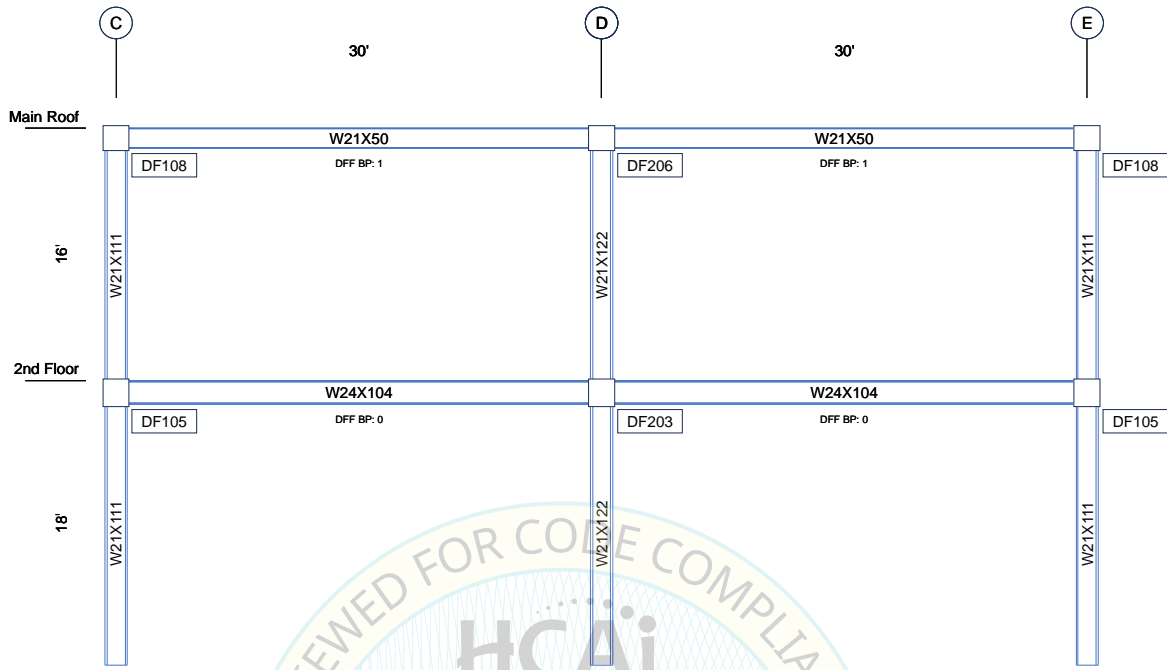
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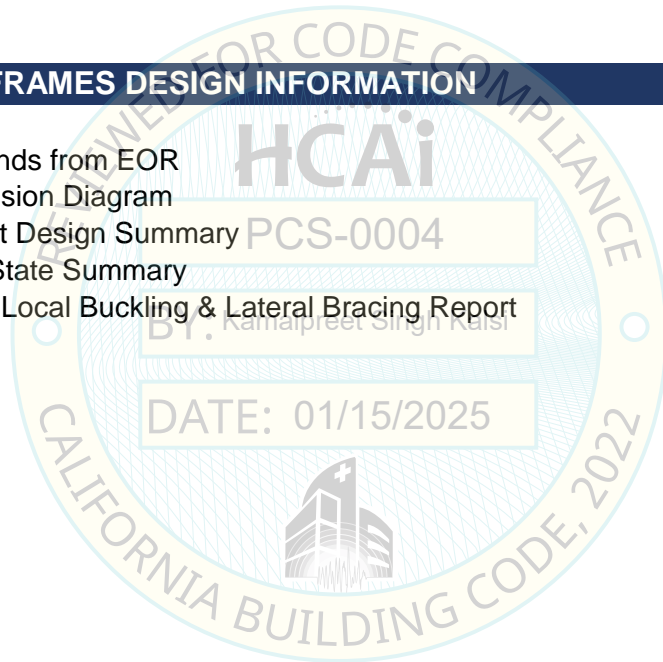






2 DURAFUSE FRAMES DESIGN INFORMATION

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DEMANDS FROM EOR HCAI CALCULATIONS

The following two tables summarize the demands from the EOR that were used for DuraFuse Frames connection design. Applicable demands for the DuraFuse connection include in-plane gravity shear, orthogonal gravity shear, orthogonal gravity moment, column axial compression, in-plane beam moment, in-plane cantilever beam moment, and collector forces. These demands are referenced in the connection design calculations.

The two tables that summarize the demands are: 1) Gravity and Earthquake demand summary and 2) Wind demand summary. Demands are obtained from the Engineer of Record and are factored according to ASCE 7 (see combinations below). Gravity demands are from the maximum of load combinations 1, 2, and 3. Earthquake demands are from load combination 5 and wind demands are from load combination 4. Other load combinations do not govern DuraFuse connection design but must be considered by the Engineer of Record, as required by ASCE 7 and CBC.

An ID has been assigned to each connection in the building (e.g., M101, see Column 1). Connections are then grouped under a global ID, indicated by the bold ID (e.g., **DF101**). These are the DuraFuse connection ID's shown in the schedules and drawings. The final demands used to design each connection ID are the maximum and bold values for grouped designs. The highlighted loads indicate where the controlling load came from. This approach is conservative because it is combining maximum demands from different joints in the group.

For the columns labeled "Orthogonal Gravity Shear" and "Moment", the maximum demand of the two possible orthogonal beams is provided.

- Load Combination 1: 1.4D
- Load Combination 2: 1.2D + 1.6L + 0.5(Lr or S or R)
- Load Combination 3: 1.2D + 1.6(Lr or S or R) + (L or 0.5W)
- Load Combination 4: 1.2D + 1.0W + LL + 0.5(LLr or S or R)
- Load Combination 5: (1.2 + 0.2SDS) D + pQE + L + 0.2S

* A minimum of 0.1A_sF_y is used in collector design unless the demand from EOR exceeds this minimum value.

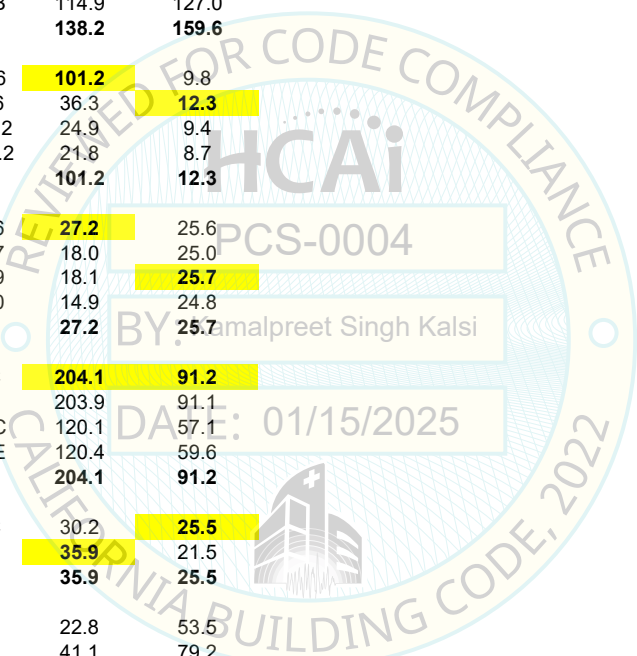
GRAVITY & EARTHQUAKE DEMANDS			LC 1,2,3	LC 1,2,3	LC 1,2,3	LC 1,2,3	LC 5	LC 5	LC 1,2,3,5	
Model ID DF ID	Framing Level	Grid	DFF Beam 1 Gravity Shear (kips)	Beam 2 Gravity Shear (SMF or gravity beam) (kips)	Orthogonal Gravity Shear (kips)	Orthogonal Gravity Moment (k-ft)	Column Axial Compression (kips)	DFF Beam 1 Moment @ Face of Column (k-ft)	Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	In-Plane Collector Force (From EOR) (kips) *
M101	2nd Floor	2 - C	43.8	43.8	90.7		345.3	550.5		
M102	2nd Floor	2 - E	40.1	40.1	86.2		340.7	548.8		
DF101			43.8	43.8	90.7		345.3	550.5		
M103	2nd Floor	3 - B	58.1	50.3	46.5		203.3	423.4		
M104	2nd Floor	3 - F	48.8	45.8	40.8		196.5	406.4		
M105	2nd Floor	5 - B	50.3	50.3	50.3		190.3	407.5		
M106	2nd Floor	5 - F	50.1	50.3	44.1		212.5	414.6		
M110	2nd Floor	H - 17	15.7	14.7	14.7		145.8	308.6		
M114	2nd Floor	8 - F	51.0	52.1	48.1		204.9	447.9		
M115	2nd Floor	H - 19	15.7	15.7	15.7		136.4	303.1		
M117	2nd Floor	9 - B	49.4	49.4	49.4		185.5	390.1		
M118	2nd Floor	9 - F	51.0	46.5	42.4		188.5	445.6		
M122	2nd Floor	11 - B	50.1	50.1	50.1		188.8	398.2		
DF102			58.1	50.3	50.3		212.5	447.9		
M107	2nd Floor	H - 16	5.4		14.7		108.7	66.7		
M108	2nd Floor	J - 16	5.4	23.6	22.6		46.9	69.0		
M120	2nd Floor	J - 21.2	31.5		31.5		33.5	58.7		
M121	2nd Floor	K - 21.2	4.3		26.3		30.3	58.2		
DF103			31.5		31.5		108.7	69.0		
M109	2nd Floor	L - 16	27.6		27.6		33.9	61.4		
M111	2nd Floor	L - 17	8.4	13.2	4.6		24.2	61.0		
M116	2nd Floor	L - 19	8.4	13.3	4.6		24.5	62.7		
M119	2nd Floor	L - 20	8.4	8.3	4.6		21.5	61.9		
DF104			27.6		27.6		33.9	62.7		
M112	2nd Floor	7 - C	45.9	42.4	89.5		303.6	445.8		
M113	2nd Floor	7 - E	45.9	45.9	89.5		303.6	445.9		
M123	2nd Floor	12 - C	27.8	26.8	89.5		209.4	385.9		
M124	2nd Floor	12 - E	27.8	27.8	89.5		209.8	388.5		
DF105			45.9	42.4	89.5		303.6	445.9		

GRAVITY & EARTHQUAKE DEMANDS (Continued)

Model ID DF ID	Framing Level	Grid	DFF Beam 1 Gravity Shear (kips)	Beam 2 Gravity Shear (SMF or gravity beam) (kips)	Orthogonal Gravity Shear (kips)	Orthogonal Gravity Moment (k-ft)	Column Axial Compression (kips)	DFF Beam 1 Moment @ Face of Column (k-ft)	Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	In-Plane Collector Force (From EOR) (kips) *
M125	Main Roof	2 - C	7.3	4.1	28.9	115.7	69.5	222.1		
M126	Main Roof	2 - E	7.3	7.8	36.7	146.9	75.8	220.4		
DF106			7.3	4.1	36.7	146.9	75.8	222.1		
M127	Main Roof	3 - B	16.4		14.4		38.6	132.9		
M128	Main Roof	3 - F	25.2	33.5	7.8		54.9	153.2		
M129	Main Roof	5 - B	16.2	16.2	16.2		39.6	133.3		
M130	Main Roof	5 - F	34.1	33.9	7.3		60.7	167.0		
M131	Main Roof	H - 17	29.9	19.2	29.9	239.1	65.6	153.9		
M134	Main Roof	8 - F	18.5	19.6	7.3		43.0	138.1		
M135	Main Roof	H - 19	12.5	28.1	28.1	224.8	56.4	141.2		
M136	Main Roof	9 - B	14.8	12.7	12.7		38.2	122.7		
M137	Main Roof	9 - F	16.9	17.1	7.3		35.4	133.6		
M138	Main Roof	11 - B	14.8	14.8	14.8		40.0	119.5		
DF107			34.1		29.9	239.1	65.6	167.0		
M132	Main Roof	7 - C	8.7	7.3	22.7		59.7	132.8		
M133	Main Roof	7 - E	8.7	8.7	24.2		62.6	133.0		
M139	Main Roof	12 - C	7.3	7.2	23.0		46.2	118.1		
M140	Main Roof	12 - E	7.3	7.3	23.0		46.4	120.4		
DF108			8.7	7.3	24.2		62.6	133.0		
M201	2nd Floor	2 - D	40.1	40.1	89.5		224.7	601.7	598.2	
DF201			40.1	40.1	89.5		224.7	601.7	598.2	
M202	2nd Floor	4 - B	52.7	52.7	52.7		135.6	507.1	501.4	
M203	2nd Floor	4 - F	49.5	50.1	44.1		156.7	489.6	489.7	
M205	2nd Floor	H - 18	15.7	15.7	15.7		69.5	315.9	318.3	
M206	2nd Floor	10 - B	45.0	50.1	45.0		121.6	468.4	485.6	
DF202			52.7	52.7	52.7		156.7	507.1	501.4	
M204	2nd Floor	7 - D	45.9	45.9	89.5		224.1	470.1	469.9	
M207	2nd Floor	12 - D	27.8	27.8	89.5		129.0	389.2	391.3	
DF203			45.9	45.9	89.5		224.1	470.1	469.9	
M208	Main Roof	2 - D	7.8	7.3	36.6	146.4	38.0	217.0	213.1	
DF204			7.8	7.3	36.6	146.4	38.0	217.0	213.1	
M209	Main Roof	4 - B	16.4	17.1	7.3		28.5	139.2	145.0	
M210	Main Roof	4 - F	39.7	27.0	7.3		52.2	216.5	193.8	
M212	Main Roof	H - 18	15.8	28.7	28.7	229.6	40.3	165.4	158.2	
M213	Main Roof	10 - B	14.8	14.8	14.8		26.1	127.6	130.0	
DF205			39.7	28.7	7.3		52.2	216.5	193.8	
M211	Main Roof	7 - D	8.7	8.7	24.6		38.8	128.9	129.1	
M214	Main Roof	12 - D	7.3	7.3	23.1		26.8	121.2	119.6	
DF206			8.7	8.7	24.6		38.8	128.9	129.1	

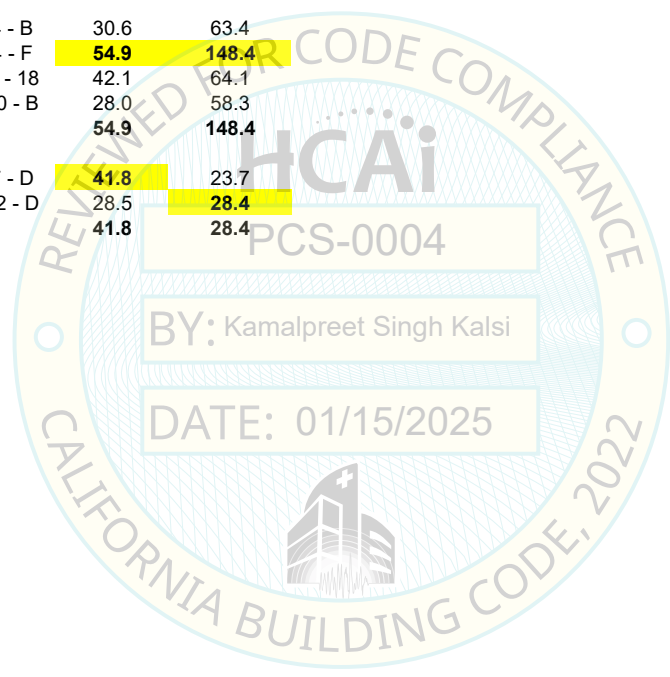
WIND DEMANDS

Model ID DF ID	Framing Level	Grid	LC 4 Column Axial Compression (kips)	LC 4 DFF Beam 1 Moment @ Face of Column (k-ft)	LC 4 Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	LC 4 In-Plane Collector Force (From EOR) (kips) *
M101	2nd Floor	2 - C	201.5	86.9		
M102	2nd Floor	2 - E	196.8	83.6		
DF101			201.5	86.9		
M103	2nd Floor	3 - B	125.2	143.6		
M104	2nd Floor	3 - F	122.2	129.8		
M105	2nd Floor	5 - B	112.9	127.5		
M106	2nd Floor	5 - F	138.2	137.2		
M110	2nd Floor	H - 17	70.8	47.6		
M114	2nd Floor	8 - F	125.5	159.6		
M115	2nd Floor	H - 19	60.7	41.5		
M117	2nd Floor	9 - B	111.5	118.6		
M118	2nd Floor	9 - F	109.5	156.2		
M122	2nd Floor	11 - B	114.9	127.0		
DF102			138.2	159.6		
M107	2nd Floor	H - 16	101.2	9.8		
M108	2nd Floor	J - 16	36.3	12.3		
M120	2nd Floor	J - 21.2	24.9	9.4		
M121	2nd Floor	K - 21.2	21.8	8.7		
DF103			101.2	12.3		
M109	2nd Floor	L - 16	27.2	25.6		
M111	2nd Floor	L - 17	18.0	25.0		
M116	2nd Floor	L - 19	18.1	25.7		
M119	2nd Floor	L - 20	14.9	24.8		
DF104			27.2	25.7		
M112	2nd Floor	7 - C	204.1	91.2		
M113	2nd Floor	7 - E	203.9	91.1		
M123	2nd Floor	12 - C	120.1	57.1		
M124	2nd Floor	12 - E	120.4	59.6		
DF105			204.1	91.2		
M125	Main Roof	2 - C	30.2	25.5		
M126	Main Roof	2 - E	35.9	21.5		
DF106			35.9	25.5		
M127	Main Roof	3 - B	22.8	53.5		
M128	Main Roof	3 - F	41.1	79.2		
M129	Main Roof	5 - B	23.9	53.1		
M130	Main Roof	5 - F	47.8	95.1		
M131	Main Roof	H - 17	44.7	48.7		
M134	Main Roof	8 - F	28.4	62.4		
M135	Main Roof	H - 19	35.1	35.6		
M136	Main Roof	9 - B	24.0	49.9		
M137	Main Roof	9 - F	20.9	56.9		
M138	Main Roof	11 - B	25.9	47.6		
DF107			47.8	95.1		
M132	Main Roof	7 - C	39.3	23.4		
M133	Main Roof	7 - E	41.9	23.4		
M139	Main Roof	12 - C	27.6	22.4		
M140	Main Roof	12 - E	27.8	24.6		
DF108			41.9	24.6		
M201	2nd Floor	2 - D	215.8	160.5		
DF201			215.8	160.5		



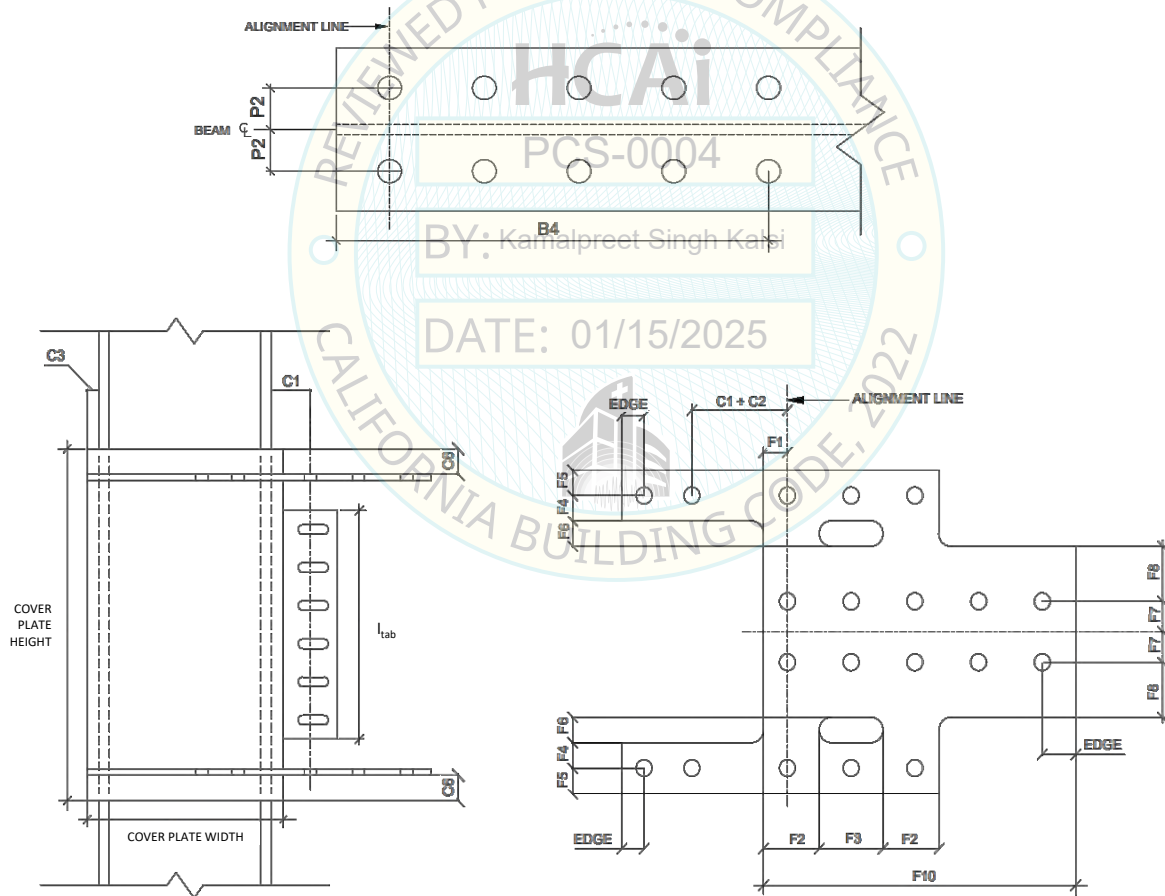
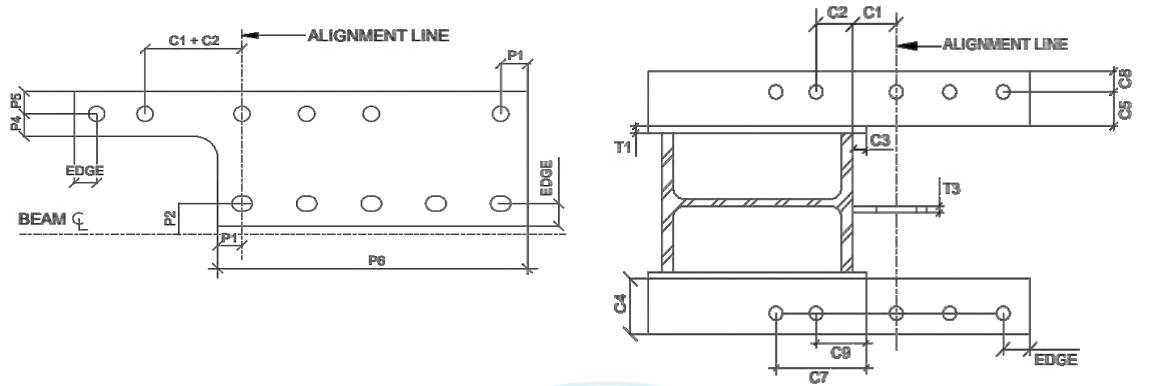
WIND DEMANDS (Continued)

Model ID DF ID	Framing Level	Grid	Column Axial Compression (kips)	DFF Beam 1 Moment @ Face of Column (k-ft)	Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	In-Plane Collector Force (From EOR) (kips) *
M202	2nd Floor	4 - B	132.4	232.1		
M203	2nd Floor	4 - F	153.9	217.4		
M205	2nd Floor	H - 18	72.4	66.6		
M206	2nd Floor	10 - B	118.6	202.9		
DF202			153.9	232.1		
M204	2nd Floor	7 - D	218.4	144.8		
M207	2nd Floor	12 - D	125.8	89.2		
DF203			218.4	144.8		
M208	Main Roof	2 - D	39.2	31.3		
DF204			39.2	31.3		
M209	Main Roof	4 - B	30.6	63.4		
M210	Main Roof	4 - F	54.9	148.4		
M212	Main Roof	H - 18	42.1	64.1		
M213	Main Roof	10 - B	28.0	58.3		
DF205			54.9	148.4		
M211	Main Roof	7 - D	41.8	23.7		
M214	Main Roof	12 - D	28.5	28.4		
DF206			41.8	28.4		



DIMENSION DIAGRAMS

HCAI CALCULATIONS

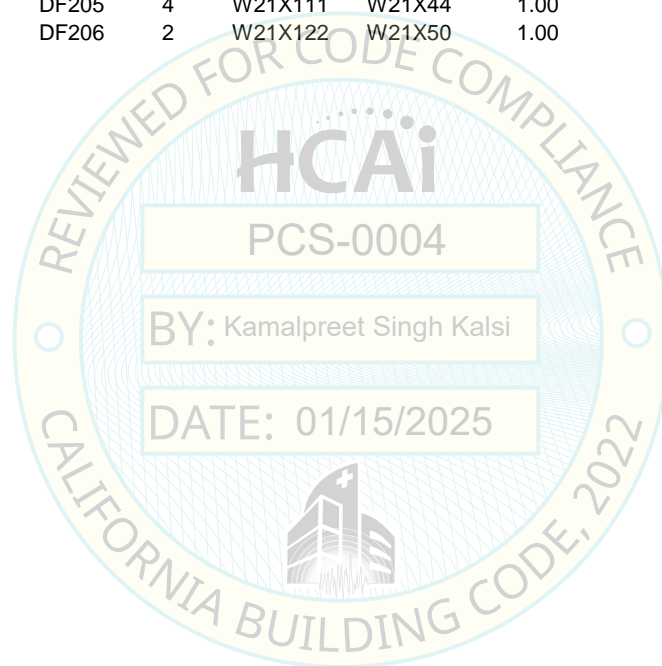


REVIEWED FOR CODE COMPLIANCE
HCAi
 PCS-0004
 BY: Kamalpreet Singh Kalsi
 DATE: 01/15/2025
 CALIFORNIA BUILDING CODE, 2022

PROJECT SUMMARY

HCAI CALCULATIONS

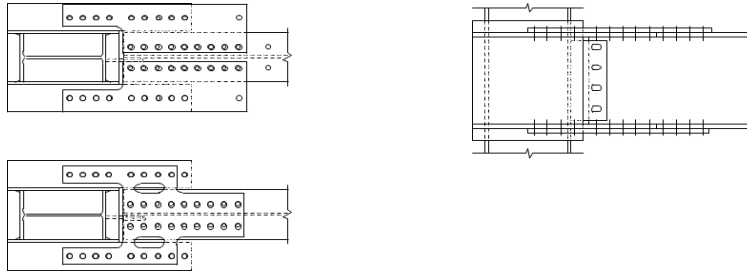
ID	Count	Column	Beam	Controlling D/C Ratio
DF101	2	W21X122	W24X146	1.00
DF102	10	W21X73	W24X94	1.00
DF103	4	W16X57	W24X55	1.00
DF104	4	W16X50	W18X35	1.00
DF105	4	W21X111	W24X104	1.00
DF106	2	W21X122	W24X131	1.00
DF107	10	W21X73	W21X44	1.00
DF108	4	W21X111	W21X50	1.00
DF201	1	W21X166	W24X146	1.00
DF202	4	W21X111	W24X94	1.00
DF203	2	W21X122	W24X104	1.00
DF204	1	W21X166	W24X131	1.00
DF205	4	W21X111	W21X44	1.00
DF206	2	W21X122	W21X50	1.00



See following pages for more information

DF101 Limit State Summary

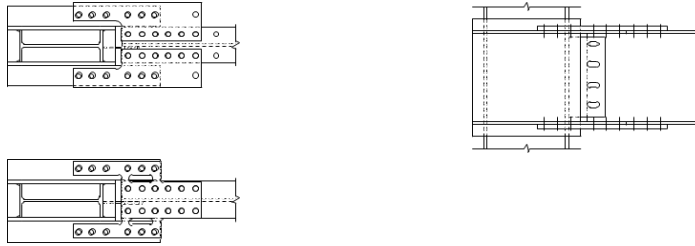
Column: W21X122 - Beam: W24X146



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 5.9$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 6.4$	OK
	Column-beam moment ratio	$\Sigma M^*_{pc} = 3.07E+04$	$\Sigma M^*_{pb} = 2.27E+04$	OK
2	Maximum probable moment	$M_u = 6606$ k-in	$M_{pr} = 20900$ k-in	0.32
3	Cover plate shear yielding	$R_u = 512.1$ kips	$\phi R_n = 634.5$ kips	0.81
	Cover plate thickness	$t_{cp} = 0.75$ in	$t_{req} = 0.47$ in	OK
4	Beam net section check	$M_{pe} = 22990$ k-in	$M_{fr} = 24827$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 512.1$ kips	$\phi R_n = 563.6$ kips	0.91
	Bolt slip for wind load	$R_u = 20.2$ kips	$\phi R_n = 207.5$ kips	0.10
6	Alignment line location, C1	$C1_{req} = 6.4$ in	$C1 = 6.4$ in	OK
7	Weld 1 strength	$r_u = 17.9$ kip/in	$\phi r_n = 19.8$ kip/in	0.90
8	Weld 2 strength	$r_u = 25.3$ kip/in	$\phi r_n = 27.8$ kip/in	0.91
9	External continuity plate rupture: Mode 1	$P_u = 284.5$ kips	$\phi P_n = 318.8$ kips	0.89
	External continuity plate rupture: Mode 2	$P_u = 341.4$ kips	$\phi P_n = 341.7$ kips	1.00
	External continuity plate rupture: Mode 3	$P_u = 398.3$ kips	$\phi P_n = 453.1$ kips	0.88
	External continuity plate rupture: Mode 4	$R_u = 341.4$ kips	$\phi R_n = 378.9$ kips	0.90
	External continuity plate rupture: Mode 5	$R_u = 398.3$ kips	$\phi R_n = 464.3$ kips	0.86
10	Beam shear	$V_u = 167.4$ kips	$\phi V_n = 481.6$ kips	0.35
11	Beam block shear	$R_u = 1024.3$ kips	$\phi R_n = 1437.4$ kips	0.71
12	Web bolt shear	$R_u = 167.4$ kips	$\phi R_n = 250.5$ kips	0.67
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.35
	Shear tab weld failure	$r_u = 10.4$ kips	$\phi r_n = 11.1$ kips	0.93
15	Top plate for shear yielding	$R_u = 512.1$ kips	$\phi R_n = 1046.2$ kips	0.49
	Top plate for shear rupture	$R_u = 512.1$ kips	$\phi R_n = 575.9$ kips	0.89
16	Top plate for tensile rupture in extensions	$P_u = 227.6$ kips	$\phi P_n = 233.1$ kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -6.4$ in	$P2 = 2.8$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 2.4$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 346.8$ kips	$\phi P_n = 551.8$ kips	0.63
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 346.8$ kips	$\phi P_n = 519.9$ kips	0.67
19	Fuse yielding region dimension	$F2_{max} = 3.1$ in	$F2 = 3.1$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 0.9$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 346.8$ kips	$\phi R_n = 430.3$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 346.8$ kips	$\phi R_n = 362$ kips	0.96
Misc	Bridge plate tension failure	$P_u = 69.3$ kips	$\phi P_n = 101.2$ kips	0.68
	Bridge plate to cover plate weld failure	$R_u = 69.3$ kips	$\phi R_n = 75.2$ kips	0.92
	Shear tab to bridge plate weld failure	$R_u = 43.2$ kips	$\phi R_n = 45.2$ kips	0.96
	Bridge plate to column flange weld failure	$R_u = 43.2$ kips	$\phi R_n = 67.9$ kips	0.64
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
External continuity plates with out-of-plane forces				N/A
Controlling Demand-Capacity Ratio				1.00

DF102 Limit State Summary

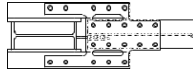
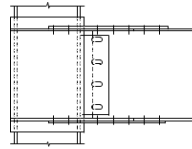
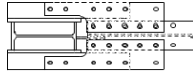
Column: W21X73 - Beam: W24X94



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 5.2$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 5.6$	OK
	Column-beam moment ratio	$\Sigma M_{pc}^* = 1.70E+04$	$\Sigma M_{pb}^* = 1.41E+04$	OK
2	Maximum probable moment	$M_u = 5374$ k-in	$M_{pr} = 12700$ k-in	0.42
3	Cover plate shear yielding	$R_u = 320.2$ kips	$\phi R_n = 612$ kips	0.52
	Cover plate thickness	$t_{cp} = 0.75$ in	$t_{req} = 0.47$ in	OK
4	Beam net section check	$M_{pe} = 13970$ k-in	$M_{fr} = 14131$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 320.2$ kips	$\phi R_n = 375.7$ kips	0.85
	Bolt slip for wind load	$R_u = 37.9$ kips	$\phi R_n = 138.3$ kips	0.27
6	Alignment line location, C1	$C1_{req} = 6.1$ in	$C1 = 6.1$ in	OK
7	Weld 1 strength	$r_u = 11$ kip/in	$\phi r_n = 11.9$ kip/in	0.93
8	Weld 2 strength	$r_u = 15.5$ kip/in	$\phi r_n = 16.7$ kip/in	0.93
9	External continuity plate rupture: Mode 1	$P_u = 160.1$ kips	$\phi P_n = 192$ kips	0.83
	External continuity plate rupture: Mode 2	$P_u = 213.5$ kips	$\phi P_n = 218.4$ kips	0.98
	External continuity plate rupture: Mode 3	$P_u = 266.9$ kips	$\phi P_n = 338.3$ kips	0.79
	External continuity plate rupture: Mode 4	$R_u = 213.5$ kips	$\phi R_n = 248.2$ kips	0.86
	External continuity plate rupture: Mode 5	$R_u = 266.9$ kips	$\phi R_n = 324.1$ kips	0.82
10	Beam shear	$V_u = 133.1$ kips	$\phi V_n = 375.4$ kips	0.35
11	Beam block shear	$R_u = 640.5$ kips	$\phi R_n = 673.8$ kips	0.95
12	Web bolt shear	$R_u = 133.1$ kips	$\phi R_n = 250.5$ kips	0.53
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.43
	Shear tab weld failure	$r_u = 9.4$ kips	$\phi r_n = 11.1$ kips	0.85
15	Top plate for shear yielding	$R_u = 320.2$ kips	$\phi R_n = 611.2$ kips	0.52
	Top plate for shear rupture	$R_u = 320.2$ kips	$\phi R_n = 332.7$ kips	0.96
16	Top plate for tensile rupture in extensions	$P_u = 160.1$ kips	$\phi P_n = 164.5$ kips	0.97
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -1.9$ in	$P2 = 2.8$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 1.5$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 200.8$ kips	$\phi P_n = 396$ kips	0.51
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 200.8$ kips	$\phi P_n = 339.3$ kips	0.59
19	Fuse yielding region dimension	$F2_{max} = 2$ in	$F2 = 2$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 0.8$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 200.8$ kips	$\phi R_n = 315$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 200.8$ kips	$\phi R_n = 234$ kips	0.86
Misc	Bridge plate tension failure	$P_u = 38$ kips	$\phi P_n = 56.2$ kips	0.68
	Bridge plate to cover plate weld failure	$R_u = 38$ kips	$\phi R_n = 41.8$ kips	0.91
	Shear tab to bridge plate weld failure	$R_u = 33.5$ kips	$\phi R_n = 41.8$ kips	0.80
	Bridge plate to column flange weld failure	$R_u = 33.5$ kips	$\phi R_n = 50.1$ kips	0.67
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF103 Limit State Summary

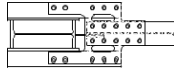
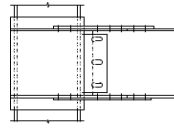
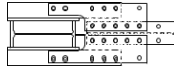
Column: W16X57 - Beam: W24X55



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 6.9$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 5$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 828$ k-in	$M_{pr} = 5895$ k-in	0.14
3	Cover plate shear yielding	$R_u = 162.2$ kips	$\phi R_n = 414$ kips	0.39
	Cover plate thickness	$t_{cp} = 0.75$ in	$t_{req} = 0.417$ in	OK
4	Beam net section check	$M_{pe} = 7370$ k-in	$M_{fr} = 7809$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 162.2$ kips	$\phi R_n = 189.4$ kips	0.86
	Bolt slip for wind load	$R_u = 3$ kips	$\phi R_n = 70.6$ kips	0.04
6	Alignment line location, C1	$C1_{req} = 4.2$ in	$C1 = 2.6$ in	OK
7	Weld 1 strength	$r_u = 9.4$ kip/in	$\phi r_n = 9.9$ kip/in	0.95
8	Weld 2 strength	$r_u = 15.3$ kip/in	$\phi r_n = 16.7$ kip/in	0.91
9	External continuity plate rupture: Mode 1	$P_u = 97.3$ kips	$\phi P_n = 99$ kips	0.98
	External continuity plate rupture: Mode 2	$P_u = 129.7$ kips	$\phi P_n = 141.9$ kips	0.91
	External continuity plate rupture: Mode 3	$P_u = 162.2$ kips	$\phi P_n = 276.2$ kips	0.59
	External continuity plate rupture: Mode 4	$R_u = 129.7$ kips	$\phi R_n = 151.8$ kips	0.85
	External continuity plate rupture: Mode 5	$R_u = 162.2$ kips	$\phi R_n = 206.2$ kips	0.79
10	Beam shear	$V_u = 65.9$ kips	$\phi V_n = 279.7$ kips	0.24
11	Beam block shear	$R_u = 324.4$ kips	$\phi R_n = 391.1$ kips	0.83
12	Web bolt shear	$R_u = 65.9$ kips	$\phi R_n = 151.5$ kips	0.43
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.07
	Shear tab weld failure	$r_u = 13.3$ kips	$\phi r_n = 13.9$ kips	0.95
15	Top plate for shear yielding	$R_u = 162.2$ kips	$\phi R_n = 360.9$ kips	0.45
	Top plate for shear rupture	$R_u = 162.2$ kips	$\phi R_n = 249.1$ kips	0.65
16	Top plate for tensile rupture in extensions	$P_u = 64.9$ kips	$\phi P_n = 68.6$ kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -5.3$ in	$P2 = 2.4$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 2.4$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 97.3$ kips	$\phi P_n = 207.3$ kips	0.47
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 97.3$ kips	$\phi P_n = 187.2$ kips	0.52
19	Fuse yielding region dimension	$F2_{max} = 1.7$ in	$F2 = 1.6$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 0.9$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 97.3$ kips	$\phi R_n = 161.7$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 97.3$ kips	$\phi R_n = 128$ kips	0.76
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	$K_{req} = 1.96E+06$	$K_S = 5.59E+06$	OK
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF104 Limit State Summary

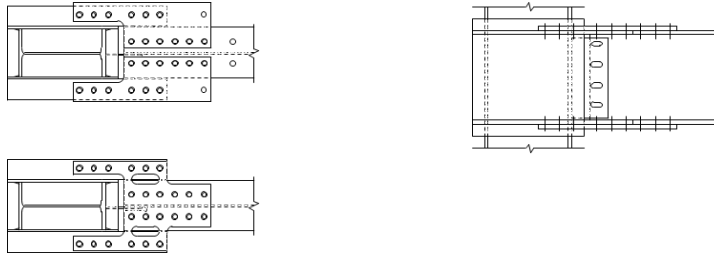
Column: W16X50 - Beam: W18X35



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 7.1$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 5.6$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 752$ k-in	$M_{pr} = 3325$ k-in	0.23
3	Cover plate shear yielding	$R_u = 117.1$ kips	$\phi R_n = 274.5$ kips	0.43
	Cover plate thickness	$t_{cp} = 0.5$ in	$t_{req} = 0.354$ in	OK
4	Beam net section check	$M_{pe} = 3658$ k-in	$M_{fr} = 3639$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 117.1$ kips	$\phi R_n = 189.4$ kips	0.62
	Bolt slip for wind load	$R_u = 8.5$ kips	$\phi R_n = 70.6$ kips	0.12
6	Alignment line location, C1	$C1_{req} = 4.4$ in	$C1 = 2.2$ in	OK
7	Weld 1 strength	$r_u = 7.5$ kip/in	$\phi r_n = 9.8$ kip/in	0.77
8	Weld 2 strength	$r_u = 12$ kip/in	$\phi r_n = 13.9$ kip/in	0.86
9	External continuity plate rupture: Mode 1	$P_u = 70.3$ kips	$\phi P_n = 73.1$ kips	0.96
	External continuity plate rupture: Mode 2	$P_u = 93.7$ kips	$\phi P_n = 103$ kips	0.91
	External continuity plate rupture: Mode 3	$P_u = 117.1$ kips	$\phi P_n = 176.1$ kips	0.66
	External continuity plate rupture: Mode 4	$R_u = 93.7$ kips	$\phi R_n = 112.5$ kips	0.83
	External continuity plate rupture: Mode 5	$R_u = 117.1$ kips	$\phi R_n = 146.2$ kips	0.80
10	Beam shear	$V_u = 46.9$ kips	$\phi V_n = 159.3$ kips	0.29
11	Beam block shear	$R_u = 234.2$ kips	$\phi R_n = 241.9$ kips	0.97
12	Web bolt shear	$R_u = 46.9$ kips	$\phi R_n = 92.1$ kips	0.51
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.15
	Shear tab weld failure	$r_u = 16.1$ kips	$\phi r_n = 16.7$ kips	0.96
15	Top plate for shear yielding	$R_u = 117.1$ kips	$\phi R_n = 240$ kips	0.49
	Top plate for shear rupture	$R_u = 117.1$ kips	$\phi R_n = 151.7$ kips	0.77
16	Top plate for tensile rupture in extensions	$P_u = 46.8$ kips	$\phi P_n = 48.8$ kips	0.96
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -2.1$ in	$P2 = 1.9$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t)_{max} = 4.2$	$F6/t_p = 3.2$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 73.1$ kips	$\phi P_n = 142.2$ kips	0.51
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 73.1$ kips	$\phi P_n = 119$ kips	0.61
19	Fuse yielding region dimension	$F2_{max} = 1.7$ in	$F2 = 1.6$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 73.1$ kips	$\phi R_n = 135$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 73.1$ kips	$\phi R_n = 109.7$ kips	0.67
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF105 Limit State Summary

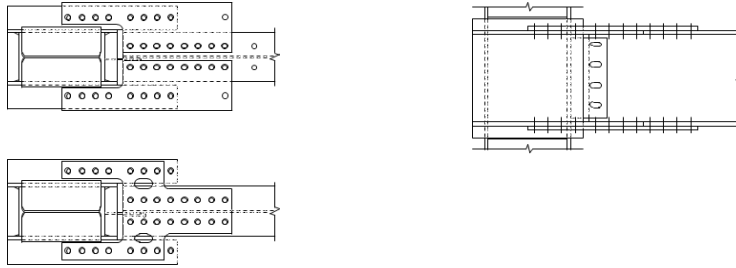
Column: W21X111 - Beam: W24X104



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 8.5$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 7$	OK
	Column-beam moment ratio	$\Sigma M_{pc}^* = 2.80E+04$	$\Sigma M_{pb}^* = 1.59E+04$	OK
2	Maximum probable moment	$M_u = 5350$ k-in	$M_{pr} = 14450$ k-in	0.37
3	Cover plate shear yielding	$R_u = 363.2$ kips	$\phi R_n = 412.5$ kips	0.88
	Cover plate thickness	$t_{cp} = 0.5$ in	$t_{req} = 0.471$ in	OK
4	Beam net section check	$M_{pe} = 15895$ k-in	$M_{fr} = 17220$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 363.2$ kips	$\phi R_n = 375.7$ kips	0.97
	Bolt slip for wind load	$R_u = 21.7$ kips	$\phi R_n = 138.3$ kips	0.16
6	Alignment line location, C1	$C1_{req} = 6.1$ in	$C1 = 6.1$ in	OK
7	Weld 1 strength	$r_u = 12.7$ kip/in	$\phi r_n = 14.1$ kip/in	0.90
8	Weld 2 strength	$r_u = 18$ kip/in	$\phi r_n = 19.5$ kip/in	0.92
9	External continuity plate rupture: Mode 1	$P_u = 181.6$ kips	$\phi P_n = 222.8$ kips	0.82
	External continuity plate rupture: Mode 2	$P_u = 242.1$ kips	$\phi P_n = 248.6$ kips	0.97
	External continuity plate rupture: Mode 3	$P_u = 302.6$ kips	$\phi P_n = 387.3$ kips	0.78
	External continuity plate rupture: Mode 4	$R_u = 242.1$ kips	$\phi R_n = 282.9$ kips	0.86
	External continuity plate rupture: Mode 5	$R_u = 302.6$ kips	$\phi R_n = 374.7$ kips	0.81
10	Beam shear	$V_u = 131.3$ kips	$\phi V_n = 361.5$ kips	0.36
11	Beam block shear	$R_u = 726.3$ kips	$\phi R_n = 768.7$ kips	0.94
12	Web bolt shear	$R_u = 131.3$ kips	$\phi R_n = 250.5$ kips	0.52
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.42
	Shear tab weld failure	$r_u = 9.6$ kips	$\phi r_n = 11.1$ kips	0.86
15	Top plate for shear yielding	$R_u = 363.2$ kips	$\phi R_n = 729.8$ kips	0.50
	Top plate for shear rupture	$R_u = 363.2$ kips	$\phi R_n = 415.4$ kips	0.87
16	Top plate for tensile rupture in extensions	$P_u = 181.6$ kips	$\phi P_n = 185.1$ kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -1.7$ in	$P2 = 2.8$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 2.1$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 229.1$ kips	$\phi P_n = 546.8$ kips	0.42
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 229.1$ kips	$\phi P_n = 513.3$ kips	0.45
19	Fuse yielding region dimension	$F2_{max} = 2.2$ in	$F2 = 2.1$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1.1$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 229.1$ kips	$\phi R_n = 354.4$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 229.1$ kips	$\phi R_n = 263.2$ kips	0.87
Misc	Bridge plate tension failure	$P_u = 44.5$ kips	$\phi P_n = 67.5$ kips	0.66
	Bridge plate to cover plate weld failure	$R_u = 44.5$ kips	$\phi R_n = 50.1$ kips	0.89
	Shear tab to bridge plate weld failure	$R_u = 33.4$ kips	$\phi R_n = 33.4$ kips	1.00
	Bridge plate to column flange weld failure	$R_u = 33.4$ kips	$\phi R_n = 50.1$ kips	0.67
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF106 Limit State Summary

Column: W21X122 - Beam: W24X131



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 6.7$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 6.4$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 2666$ k-in	$M_{pr} = 18500$ k-in	0.14
3	Cover plate shear yielding	$R_u = 457.5$ kips	$\phi R_n = 727.1$ kips	0.63
	Cover plate thickness	$t_{cp} = 0.875$ in	$t_{req} = 0.471$ in	OK
4	Beam net section check	$M_{pe} = 20350$ k-in	$M_{fr} = 22012$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 457.5$ kips	$\phi R_n = 501$ kips	0.91
	Bolt slip for wind load	$R_u = 6$ kips	$\phi R_n = 184.4$ kips	0.03
6	Alignment line location, C1	$C1_{req} = 6.1$ in	$C1 = 6.1$ in	OK
7	Weld 1 strength	$r_u = 10.8$ kip/in	$\phi r_n = 12.9$ kip/in	0.84
8	Weld 2 strength	$r_u = 21.8$ kip/in	$\phi r_n = 22.3$ kip/in	0.98
9	External continuity plate rupture: Mode 1	$P_u = 228.7$ kips	$\phi P_n = 270.8$ kips	0.84
	External continuity plate rupture: Mode 2	$P_u = 285.9$ kips	$\phi P_n = 291.4$ kips	0.98
	External continuity plate rupture: Mode 3	$P_u = 343.1$ kips	$\phi P_n = 409$ kips	0.84
	External continuity plate rupture: Mode 4	$R_u = 285.9$ kips	$\phi R_n = 324.6$ kips	0.88
	External continuity plate rupture: Mode 5	$R_u = 343.1$ kips	$\phi R_n = 410$ kips	0.84
10	Beam shear	$V_u = 116.7$ kips	$\phi V_n = 444.7$ kips	0.26
11	Beam block shear	$R_u = 915$ kips	$\phi R_n = 1150.1$ kips	0.80
12	Web bolt shear	$R_u = 116.7$ kips	$\phi R_n = 250.5$ kips	0.47
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.35
	Shear tab weld failure	$r_u = 9$ kips	$\phi r_n = 11.1$ kips	0.81
15	Top plate for shear yielding	$R_u = 457.5$ kips	$\phi R_n = 915.5$ kips	0.50
	Top plate for shear rupture	$R_u = 457.5$ kips	$\phi R_n = 497.7$ kips	0.92
16	Top plate for tensile rupture in extensions	$P_u = 228.7$ kips	$\phi P_n = 233.1$ kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -4$ in	$P2 = 2.8$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 2.2$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 361$ kips	$\phi P_n = 577.1$ kips	0.63
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 361$ kips	$\phi P_n = 552.8$ kips	0.65
19	Fuse yielding region dimension	$F2_{max} = 2.8$ in	$F2 = 2.8$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 0.9$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 361$ kips	$\phi R_n = 430.3$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 361$ kips	$\phi R_n = 362$ kips	1.00
Misc	Bridge plate tension failure	$P_u = 55.6$ kips	$\phi P_n = 78.8$ kips	0.71
	Bridge plate to cover plate weld failure	$R_u = 55.6$ kips	$\phi R_n = 58.5$ kips	0.95
	Shear tab to bridge plate weld failure	$R_u = 29.2$ kips	$\phi R_n = 33.4$ kips	0.87
	Bridge plate to column flange weld failure	$R_u = 29.2$ kips	$\phi R_n = 62.6$ kips	0.47
	Shear plate shear failure	$R_u = 192.7$ kips	$\phi R_n = 296.7$ kips	0.65
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	$Inter_{Limit} = 1.00$	$Inter = 0.55$	OK
Controlling Demand-Capacity Ratio				1.00

DF107 Limit State Summary

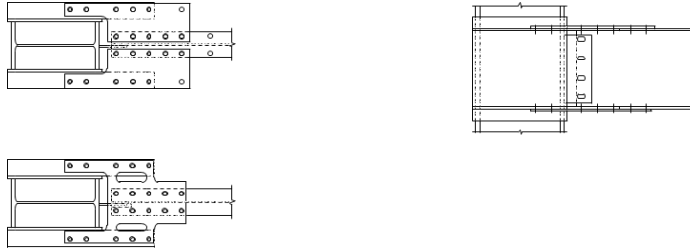
Column: W21X73 - Beam: W21X44



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 7.2$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 5.6$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 2004$ k-in	$M_{pr} = 4770$ k-in	0.42
3	Cover plate shear yielding	$R_u = 145$ kips	$\phi R_n = 348$ kips	0.42
	Cover plate thickness	$t_{cp} = 0.5$ in	$t_{req} = 0.439$ in	OK
4	Beam net section check	$M_{pe} = 5247$ k-in	$M_{fr} = 5437$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 145$ kips	$\phi R_n = 189.4$ kips	0.77
	Bolt slip for wind load	$R_u = 26.9$ kips	$\phi R_n = 70.6$ kips	0.38
6	Alignment line location, C1	$C1_{req} = 4.4$ in	$C1 = 2.4$ in	OK
7	Weld 1 strength	$r_u = 8.7$ kip/in	$\phi r_n = 9.9$ kip/in	0.88
8	Weld 2 strength	$r_u = 10.4$ kip/in	$\phi r_n = 11.1$ kip/in	0.94
9	External continuity plate rupture: Mode 1	$P_u = 87$ kips	$\phi P_n = 88.4$ kips	0.98
	External continuity plate rupture: Mode 2	$P_u = 116$ kips	$\phi P_n = 140.3$ kips	0.83
	External continuity plate rupture: Mode 3	$P_u = 145$ kips	$\phi P_n = 268.3$ kips	0.54
	External continuity plate rupture: Mode 4	$R_u = 116$ kips	$\phi R_n = 139$ kips	0.83
	External continuity plate rupture: Mode 5	$R_u = 145$ kips	$\phi R_n = 182.6$ kips	0.79
10	Beam shear	$V_u = 62.2$ kips	$\phi V_n = 217.4$ kips	0.29
11	Beam block shear	$R_u = 290$ kips	$\phi R_n = 348.3$ kips	0.83
12	Web bolt shear	$R_u = 62.2$ kips	$\phi R_n = 143.3$ kips	0.43
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.20
	Shear tab weld failure	$r_u = 15$ kips	$\phi r_n = 16.7$ kips	0.90
15	Top plate for shear yielding	$R_u = 145$ kips	$\phi R_n = 292.5$ kips	0.50
	Top plate for shear rupture	$R_u = 145$ kips	$\phi R_n = 202.9$ kips	0.71
16	Top plate for tensile rupture in extensions	$P_u = 58$ kips	$\phi P_n = 60.9$ kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -3.3$ in	$P2 = 2.1$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 3.8$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 90$ kips	$\phi P_n = 153$ kips	0.59
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 90$ kips	$\phi P_n = 133.1$ kips	0.68
19	Fuse yielding region dimension	$F2_{max} = 2.2$ in	$F2 = 2.1$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 0.9$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 90$ kips	$\phi R_n = 135$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 90$ kips	$\phi R_n = 109.7$ kips	0.82
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	$Inter_{Limit} = 1.00$	$Inter = 0.32$	OK
Controlling Demand-Capacity Ratio			1.00	

DF108 Limit State Summary

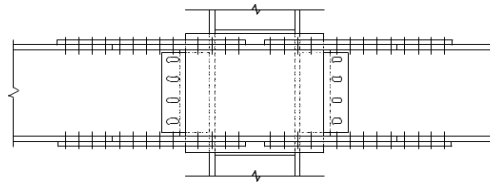
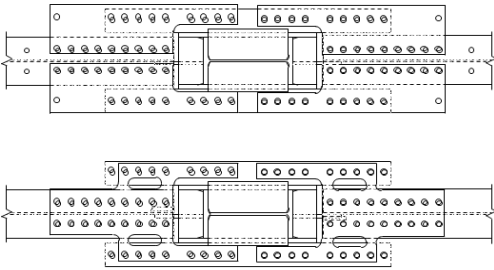
Column: W21X111 - Beam: W21X50



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 6.1$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 7$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 1596$ k-in	$M_{pr} = 5500$ k-in	0.29
3	Cover plate shear yielding	$R_u = 165.1$ kips	$\phi R_n = 352.5$ kips	0.47
	Cover plate thickness	$t_{cp} = 0.5$ in	$t_{req} = 0.439$ in	OK
4	Beam net section check	$M_{pe} = 6050$ k-in	$M_{fr} = 6218$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 165.1$ kips	$\phi R_n = 189.4$ kips	0.87
	Bolt slip for wind load	$R_u = 6.9$ kips	$\phi R_n = 70.6$ kips	0.10
6	Alignment line location, C1	$C1_{req} = 4.4$ in	$C1 = 2.4$ in	OK
7	Weld 1 strength	$r_u = 8.3$ kip/in	$\phi r_n = 9.7$ kip/in	0.86
8	Weld 2 strength	$r_u = 11.9$ kip/in	$\phi r_n = 13.9$ kip/in	0.86
9	External continuity plate rupture: Mode 1	$P_u = 99.1$ kips	$\phi P_n = 102.8$ kips	0.96
	External continuity plate rupture: Mode 2	$P_u = 132.1$ kips	$\phi P_n = 148.6$ kips	0.89
	External continuity plate rupture: Mode 3	$P_u = 165.1$ kips	$\phi P_n = 291.8$ kips	0.57
	External continuity plate rupture: Mode 4	$R_u = 132.1$ kips	$\phi R_n = 157.3$ kips	0.84
	External continuity plate rupture: Mode 5	$R_u = 165.1$ kips	$\phi R_n = 213.6$ kips	0.77
10	Beam shear	$V_u = 41.2$ kips	$\phi V_n = 237.1$ kips	0.17
11	Beam block shear	$R_u = 330.2$ kips	$\phi R_n = 430.5$ kips	0.77
12	Web bolt shear	$R_u = 41.2$ kips	$\phi R_n = 151.5$ kips	0.27
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.10
	Shear tab weld failure	$r_u = 12.6$ kips	$\phi r_n = 13.9$ kips	0.91
15	Top plate for shear yielding	$R_u = 165.1$ kips	$\phi R_n = 375$ kips	0.44
	Top plate for shear rupture	$R_u = 165.1$ kips	$\phi R_n = 262.8$ kips	0.63
16	Top plate for tensile rupture in extensions	$P_u = 66$ kips	$\phi P_n = 68.6$ kips	0.96
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -3.8$ in	$P2 = 2.1$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 3.4$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 102.7$ kips	$\phi P_n = 296.7$ kips	0.35
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 102.7$ kips	$\phi P_n = 303.5$ kips	0.34
19	Fuse yielding region dimension	$F2_{max} = 2$ in	$F2 = 1.9$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1.1$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 102.7$ kips	$\phi R_n = 168.8$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 102.7$ kips	$\phi R_n = 137.1$ kips	0.75
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF201 Limit State Summary

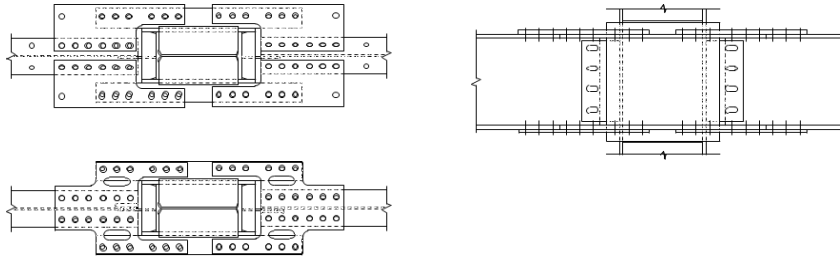
Column: W21X166 - Beam: W24X146



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 5.9$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 4.6$	OK
	Column-beam moment ratio	$\Sigma M_{pc}^* = 4.87E+04$	$\Sigma M_{pb}^* = 4.55E+04$	OK
2	Maximum probable moment	$M_u = 7221$ k-in	$M_{pr} = 20900$ k-in	0.35
3	Cover plate shear yielding	$R_u = 912.9$ kips	$\phi R_n = 1164.4$ kips	0.78
	Cover plate thickness	$t_{cp} = 1.125$ in	$t_{req} = 0.47$ in	OK
4	Beam net section check	$M_{pe} = 22990$ k-in	$M_{fr} = 24827$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 510.2$ kips	$\phi R_n = 563.6$ kips	0.91
	Bolt slip for wind load	$R_u = 37.1$ kips	$\phi R_n = 207.5$ kips	0.18
6	Alignment line location, C1	$C1_{req} = 9.1$ in	$C1 = 9.1$ in	OK
7	Weld 1 strength	$r_u = 20.2$ kip/in	$\phi r_n = 21$ kip/in	0.96
8	Weld 2 strength	$r_u = 30.5$ kip/in	$\phi r_n = 33.4$ kip/in	0.91
9	External continuity plate rupture: Mode 1	$P_u = 283.4$ kips	$\phi P_n = 300.9$ kips	0.94
	External continuity plate rupture: Mode 2	$P_u = 340.1$ kips	$\phi P_n = 346.7$ kips	0.98
	External continuity plate rupture: Mode 3	$P_u = 396.8$ kips	$\phi P_n = 498.9$ kips	0.80
	External continuity plate rupture: Mode 4	$R_u = 340.1$ kips	$\phi R_n = 388.8$ kips	0.87
	External continuity plate rupture: Mode 5	$R_u = 396.8$ kips	$\phi R_n = 483.7$ kips	0.82
10	Beam shear	$V_u = 163.9$ kips	$\phi V_n = 481.6$ kips	0.34
11	Beam block shear	$R_u = 1020.4$ kips	$\phi R_n = 1437.4$ kips	0.71
12	Web bolt shear	$R_u = 163.9$ kips	$\phi R_n = 250.5$ kips	0.65
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nn}, \phi R_{nv}$ Varies	0.33
	Shear tab weld failure	$r_u = 10.4$ kips	$\phi r_n = 11.1$ kips	0.93
15	Top plate for shear yielding	$R_u = 510.2$ kips	$\phi R_n = 1153.1$ kips	0.44
	Top plate for shear rupture	$R_u = 510.2$ kips	$\phi R_n = 630.7$ kips	0.81
16	Top plate for tensile rupture in extensions	$P_u = 226.8$ kips	$\phi P_n = 228.5$ kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -9.2$ in	$P2 = 2.8$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 2.2$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 345.2$ kips	$\phi P_n = 655.3$ kips	0.53
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 345.2$ kips	$\phi P_n = 632.5$ kips	0.55
19	Fuse yielding region dimension	$F2_{max} = 2.8$ in	$F2 = 2.8$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 345.2$ kips	$\phi R_n = 435.9$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 345.2$ kips	$\phi R_n = 347.3$ kips	0.99
Misc	Bridge plate tension failure	$P_u = 58.2$ kips	$\phi P_n = 140.6$ kips	0.41
	Bridge plate to cover plate weld failure	$R_u = 58.2$ kips	$\phi R_n = 104.4$ kips	0.56
	Shear tab to bridge plate weld failure	$R_u = 60.6$ kips	$\phi R_n = 66.8$ kips	0.91
	Bridge plate to column flange weld failure	$R_u = 60.6$ kips	$\phi R_n = 86.9$ kips	0.70
	Shear plate shear failure	$R_u = 247.8$ kips	$\phi R_n = 370.9$ kips	0.67
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF202 Limit State Summary

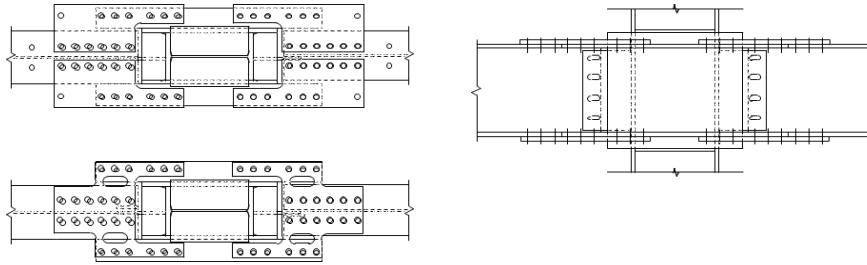
Column: W21X111 - Beam: W24X94



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 5.2$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 7$	OK
	Column-beam moment ratio	$\Sigma M^*_{pc} = 3.11E+04$	$\Sigma M^*_{pb} = 2.81E+04$	OK
2	Maximum probable moment	$M_u = 6086$ k-in	$M_{pr} = 12700$ k-in	0.48
3	Cover plate shear yielding	$R_u = 571.2$ kips	$\phi R_n = 641.2$ kips	0.89
	Cover plate thickness	$t_{cp} = 0.75$ in	$t_{req} = 0.47$ in	OK
4	Beam net section check	$M_{pe} = 13970$ k-in	$M_{fr} = 14131$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 320.2$ kips	$\phi R_n = 375.7$ kips	0.85
	Bolt slip for wind load	$R_u = 55$ kips	$\phi R_n = 138.3$ kips	0.40
6	Alignment line location, C1	$C1_{req} = 6.6$ in	$C1 = 3.1$ in	OK
7	Weld 1 strength	$r_u = 12.4$ kip/in	$\phi r_n = 13.4$ kip/in	0.92
8	Weld 2 strength	$r_u = 26.6$ kip/in	$\phi r_n = 27.8$ kip/in	0.95
9	External continuity plate rupture: Mode 1	$P_u = 160.1$ kips	$\phi P_n = 192$ kips	0.83
	External continuity plate rupture: Mode 2	$P_u = 213.5$ kips	$\phi P_n = 216.3$ kips	0.99
	External continuity plate rupture: Mode 3	$P_u = 266.9$ kips	$\phi P_n = 326.6$ kips	0.82
	External continuity plate rupture: Mode 4	$R_u = 213.5$ kips	$\phi R_n = 248.2$ kips	0.86
	External continuity plate rupture: Mode 5	$R_u = 266.9$ kips	$\phi R_n = 324.1$ kips	0.82
10	Beam shear	$V_u = 127.7$ kips	$\phi V_n = 375.4$ kips	0.34
11	Beam block shear	$R_u = 640.5$ kips	$\phi R_n = 673.8$ kips	0.95
12	Web bolt shear	$R_u = 127.7$ kips	$\phi R_n = 250.5$ kips	0.51
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.40
	Shear tab weld failure	$r_u = 9.3$ kips	$\phi r_n = 11.1$ kips	0.83
15	Top plate for shear yielding	$R_u = 320.2$ kips	$\phi R_n = 618.8$ kips	0.52
	Top plate for shear rupture	$R_u = 320.2$ kips	$\phi R_n = 340$ kips	0.94
16	Top plate for tensile rupture in extensions	$P_u = 160.1$ kips	$\phi P_n = 164.5$ kips	0.97
17	Top plate yielding due to combined flexure and shear	$P2_{req} = 0$ in	$P2 = 2.8$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 2.4$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 200.8$ kips	$\phi P_n = 508.5$ kips	0.39
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 200.8$ kips	$\phi P_n = 485.6$ kips	0.41
19	Fuse yielding region dimension	$F2_{max} = 2.2$ in	$F2 = 2.1$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1.1$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 200.8$ kips	$\phi R_n = 326.2$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 200.8$ kips	$\phi R_n = 248.6$ kips	0.81
Misc	Bridge plate tension failure	$P_u = 40$ kips	$\phi P_n = 98.4$ kips	0.41
	Bridge plate to cover plate weld failure	$R_u = 40$ kips	$\phi R_n = 73.1$ kips	0.55
	Shear tab to bridge plate weld failure	$R_u = 34.8$ kips	$\phi R_n = 39$ kips	0.89
	Bridge plate to column flange weld failure	$R_u = 34.8$ kips	$\phi R_n = 73.1$ kips	0.48
	Shear plate shear failure	$R_u = 164.9$ kips	$\phi R_n = 296.2$ kips	0.56
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF203 Limit State Summary

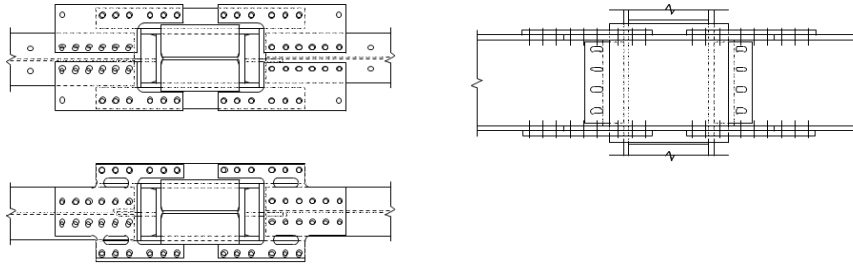
Column: W21X122 - Beam: W24X104



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 8.5$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 6.4$	OK
	Column-beam moment ratio	$\Sigma M_{pc}^* = 3.32E+04$	$\Sigma M_{pb}^* = 3.17E+04$	OK
2	Maximum probable moment	$M_u = 5641$ k-in	$M_{pr} = 14450$ k-in	0.39
3	Cover plate shear yielding	$R_u = 649.6$ kips	$\phi R_n = 884.6$ kips	0.73
	Cover plate thickness	$t_{cp} = 0.875$ in	$t_{req} = 0.471$ in	OK
4	Beam net section check	$M_{pe} = 15895$ k-in	$M_{fr} = 17220$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 363.2$ kips	$\phi R_n = 375.7$ kips	0.97
	Bolt slip for wind load	$R_u = 34.5$ kips	$\phi R_n = 138.3$ kips	0.25
6	Alignment line location, C1	$C1_{req} = 9.1$ in	$C1 = 9.1$ in	OK
7	Weld 1 strength	$r_u = 14.1$ kip/in	$\phi r_n = 15.4$ kip/in	0.92
8	Weld 2 strength	$r_u = 22.2$ kip/in	$\phi r_n = 25.1$ kip/in	0.89
9	External continuity plate rupture: Mode 1	$P_u = 181.6$ kips	$\phi P_n = 222.8$ kips	0.82
	External continuity plate rupture: Mode 2	$P_u = 242.1$ kips	$\phi P_n = 247.6$ kips	0.98
	External continuity plate rupture: Mode 3	$P_u = 302.6$ kips	$\phi P_n = 367.9$ kips	0.82
	External continuity plate rupture: Mode 4	$R_u = 242.1$ kips	$\phi R_n = 282.9$ kips	0.86
	External continuity plate rupture: Mode 5	$R_u = 302.6$ kips	$\phi R_n = 368.3$ kips	0.82
10	Beam shear	$V_u = 131.3$ kips	$\phi V_n = 361.5$ kips	0.36
11	Beam block shear	$R_u = 726.3$ kips	$\phi R_n = 727.6$ kips	1.00
12	Web bolt shear	$R_u = 131.3$ kips	$\phi R_n = 250.5$ kips	0.52
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.42
	Shear tab weld failure	$r_u = 9.6$ kips	$\phi r_n = 11.1$ kips	0.86
15	Top plate for shear yielding	$R_u = 363.2$ kips	$\phi R_n = 696.1$ kips	0.52
	Top plate for shear rupture	$R_u = 363.2$ kips	$\phi R_n = 382.5$ kips	0.95
16	Top plate for tensile rupture in extensions	$P_u = 181.6$ kips	$\phi P_n = 185.1$ kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req} = 0.4$ in	$P2 = 2.6$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 2.1$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 229.1$ kips	$\phi P_n = 589.8$ kips	0.39
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 229.1$ kips	$\phi P_n = 569.3$ kips	0.40
19	Fuse yielding region dimension	$F2_{max} = 2.2$ in	$F2 = 2.1$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1.1$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 229.1$ kips	$\phi R_n = 367$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 229.1$ kips	$\phi R_n = 279.7$ kips	0.82
Misc	Bridge plate tension failure	$P_u = 41$ kips	$\phi P_n = 112.5$ kips	0.36
	Bridge plate to cover plate weld failure	$R_u = 41$ kips	$\phi R_n = 83.5$ kips	0.49
	Shear tab to bridge plate weld failure	$R_u = 49.7$ kips	$\phi R_n = 50.1$ kips	0.99
	Bridge plate to column flange weld failure	$R_u = 49.7$ kips	$\phi R_n = 65.1$ kips	0.76
	Shear plate shear failure	$R_u = 192.7$ kips	$\phi R_n = 296.7$ kips	0.65
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
External continuity plates with out-of-plane forces	N/A	N/A	N/A	
Controlling Demand-Capacity Ratio				1.00

DF204 Limit State Summary

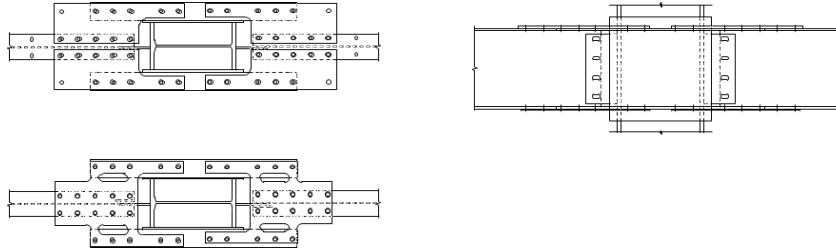
Column: W21X166 - Beam: W24X131



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 6.7$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 4.6$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 2604$ k-in	$M_{pr} = 14031$ k-in	0.19
3	Cover plate shear yielding	$R_u = 641.4$ kips	$\phi R_n = 995.6$ kips	0.64
	Cover plate thickness	$t_{cp} = 1.125$ in	$t_{req} = 0.471$ in	OK
4	Beam net section check	$M_{pe} = 20350$ k-in	$M_{fr} = 22012$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 368.9$ kips	$\phi R_n = 375.7$ kips	0.98
	Bolt slip for wind load	$R_u = 7.3$ kips	$\phi R_n = 138.3$ kips	0.05
6	Alignment line location, C1	$C1_{req} = 6.6$ in	$C1 = 6.6$ in	OK
7	Weld 1 strength	$r_u = 15.5$ kip/in	$\phi r_n = 16.2$ kip/in	0.96
8	Weld 2 strength	$r_u = 27.7$ kip/in	$\phi r_n = 30.6$ kip/in	0.91
9	External continuity plate rupture: Mode 1	$P_u = 184.5$ kips	$\phi P_n = 232.3$ kips	0.79
	External continuity plate rupture: Mode 2	$P_u = 246$ kips	$\phi P_n = 253.5$ kips	0.97
	External continuity plate rupture: Mode 3	$P_u = 307.5$ kips	$\phi P_n = 374.1$ kips	0.82
	External continuity plate rupture: Mode 4	$R_u = 246$ kips	$\phi R_n = 292.1$ kips	0.84
	External continuity plate rupture: Mode 5	$R_u = 307.5$ kips	$\phi R_n = 387$ kips	0.79
10	Beam shear	$V_u = 91$ kips	$\phi V_n = 444.7$ kips	0.20
11	Beam block shear	$R_u = 737.9$ kips	$\phi R_n = 918.4$ kips	0.80
12	Web bolt shear	$R_u = 91$ kips	$\phi R_n = 250.5$ kips	0.36
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.28
	Shear tab weld failure	$r_u = 8.5$ kips	$\phi r_n = 11.1$ kips	0.76
15	Top plate for shear yielding	$R_u = 368.9$ kips	$\phi R_n = 773.4$ kips	0.48
	Top plate for shear rupture	$R_u = 368.9$ kips	$\phi R_n = 425$ kips	0.87
16	Top plate for tensile rupture in extensions	$P_u = 184.5$ kips	$\phi P_n = 190.4$ kips	0.97
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -1.1$ in	$P2 = 2.8$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 1.8$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 218$ kips	$\phi P_n = 711.6$ kips	0.31
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 218$ kips	$\phi P_n = 705.7$ kips	0.31
19	Fuse yielding region dimension	$F2_{max} = 1.9$ in	$F2 = 1.9$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1.2$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 218$ kips	$\phi R_n = 407.8$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 218$ kips	$\phi R_n = 310.8$ kips	0.70
Misc	Bridge plate tension failure	$P_u = 46.1$ kips	$\phi P_n = 112.5$ kips	0.41
	Bridge plate to cover plate weld failure	$R_u = 46.1$ kips	$\phi R_n = 83.5$ kips	0.55
	Shear tab to bridge plate weld failure	$R_u = 24.6$ kips	$\phi R_n = 39$ kips	0.63
	Bridge plate to column flange weld failure	$R_u = 24.6$ kips	$\phi R_n = 73.1$ kips	0.34
	Shear plate shear failure	$R_u = 247.8$ kips	$\phi R_n = 370.9$ kips	0.67
	Fully Restrained (FR) assumption	$K_{req} = 5.83E+06$	$K_S = 7.07E+06$	OK
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	$Inter_{Limit} = 1.00$	$Inter = 1$	OK
Controlling Demand-Capacity Ratio				1.00

DF205 Limit State Summary

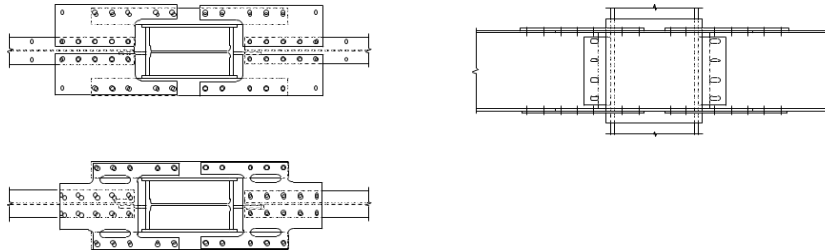
Column: W21X111 - Beam: W21X44



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 7.2$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 7$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 2598$ k-in	$M_{pr} = 4770$ k-in	0.54
3	Cover plate shear yielding	$R_u = 257.5$ kips	$\phi R_n = 382.5$ kips	0.67
	Cover plate thickness	$t_{cp} = 0.5$ in	$t_{req} = 0.439$ in	OK
4	Beam net section check	$M_{pe} = 5247$ k-in	$M_{fr} = 5437$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 145$ kips	$\phi R_n = 189.4$ kips	0.77
	Bolt slip for wind load	$R_u = 42$ kips	$\phi R_n = 70.6$ kips	0.59
6	Alignment line location, C1	$C1_{req} = 5.2$ in	$C1 = 2.4$ in	OK
7	Weld 1 strength	$r_u = 13.1$ kip/in	$\phi r_n = 14.5$ kip/in	0.90
8	Weld 2 strength	$r_u = 14.1$ kip/in	$\phi r_n = 16.7$ kip/in	0.85
9	External continuity plate rupture: Mode 1	$P_u = 87$ kips	$\phi P_n = 88.4$ kips	0.98
	External continuity plate rupture: Mode 2	$P_u = 116$ kips	$\phi P_n = 137.7$ kips	0.84
	External continuity plate rupture: Mode 3	$P_u = 145$ kips	$\phi P_n = 280.4$ kips	0.52
	External continuity plate rupture: Mode 4	$R_u = 116$ kips	$\phi R_n = 139$ kips	0.83
	External continuity plate rupture: Mode 5	$R_u = 145$ kips	$\phi R_n = 188.2$ kips	0.77
10	Beam shear	$V_u = 67.9$ kips	$\phi V_n = 217.4$ kips	0.31
11	Beam block shear	$R_u = 290$ kips	$\phi R_n = 400.9$ kips	0.72
12	Web bolt shear	$R_u = 67.9$ kips	$\phi R_n = 143.3$ kips	0.47
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.21
	Shear tab weld failure	$r_u = 16.8$ kips	$\phi r_n = 19.5$ kips	0.86
15	Top plate for shear yielding	$R_u = 145$ kips	$\phi R_n = 318.8$ kips	0.45
	Top plate for shear rupture	$R_u = 145$ kips	$\phi R_n = 228.5$ kips	0.63
16	Top plate for tensile rupture in extensions	$P_u = 58$ kips	$\phi P_n = 60.9$ kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -3.9$ in	$P2 = 2.1$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max} = 4.2$	$F6/t_p = 4.2$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 90$ kips	$\phi P_n = 243$ kips	0.37
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 90$ kips	$\phi P_n = 250.1$ kips	0.36
19	Fuse yielding region dimension	$F2_{max} = 2.3$ in	$F2 = 2.2$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 0.9$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 90$ kips	$\phi R_n = 129.4$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 90$ kips	$\phi R_n = 102.4$ kips	0.88
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

DF206 Limit State Summary

Column: W21X122 - Beam: W21X50



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange} = 9.2$	$\lambda_{flange} = 6.1$	OK
	Column flange local buckling check	$\lambda_{hd,flange} = 7.3$	$\lambda_{flange} = 6.4$	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	$M_u = 1549$ k-in	$M_{pr} = 5500$ k-in	0.28
3	Cover plate shear yielding	$R_u = 293.5$ kips	$\phi R_n = 355.5$ kips	0.83
	Cover plate thickness	$t_{cp} = 0.5$ in	$t_{req} = 0.439$ in	OK
4	Beam net section check	$M_{pe} = 6050$ k-in	$M_{fr} = 6218$ k-in	OK
5	Beam flange bolt (bearing and tearout) failure	$R_u = 165.1$ kips	$\phi R_n = 189.4$ kips	0.87
	Bolt slip for wind load	$R_u = 8$ kips	$\phi R_n = 70.6$ kips	0.11
6	Alignment line location, C1	$C1_{req} = 4.1$ in	$C1 = 2.4$ in	OK
7	Weld 1 strength	$r_u = 15.7$ kip/in	$\phi r_n = 16.9$ kip/in	0.93
8	Weld 2 strength	$r_u = 24$ kip/in	$\phi r_n = 25.1$ kip/in	0.96
9	External continuity plate rupture: Mode 1	$P_u = 99.1$ kips	$\phi P_n = 102.8$ kips	0.96
	External continuity plate rupture: Mode 2	$P_u = 132.1$ kips	$\phi P_n = 144.4$ kips	0.91
	External continuity plate rupture: Mode 3	$P_u = 165.1$ kips	$\phi P_n = 280.1$ kips	0.59
	External continuity plate rupture: Mode 4	$R_u = 132.1$ kips	$\phi R_n = 157.3$ kips	0.84
	External continuity plate rupture: Mode 5	$R_u = 165.1$ kips	$\phi R_n = 215.3$ kips	0.77
10	Beam shear	$V_u = 41.2$ kips	$\phi V_n = 237.1$ kips	0.17
11	Beam block shear	$R_u = 330.2$ kips	$\phi R_n = 446.1$ kips	0.74
12	Web bolt shear	$R_u = 41.2$ kips	$\phi R_n = 151.5$ kips	0.27
13	Shear tab rupture through net section	P_u, V_u Varies	$\phi R_{nt}, \phi R_{nv}$ Varies	0.10
	Shear tab weld failure	$r_u = 12.4$ kips	$\phi r_n = 13.9$ kips	0.89
15	Top plate for shear yielding	$R_u = 165.1$ kips	$\phi R_n = 379.7$ kips	0.43
	Top plate for shear rupture	$R_u = 165.1$ kips	$\phi R_n = 267.4$ kips	0.62
16	Top plate for tensile rupture in extensions	$P_u = 66$ kips	$\phi P_n = 68.6$ kips	0.96
17	Top plate yielding due to combined flexure and shear	$P2_{req} = -3.9$ in	$P2 = 2.1$ in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t)_{max} = 4.2$	$F6/t_p = 3.4$	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp} = 102.7$ kips	$\phi P_n = 320.6$ kips	0.32
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp} = 102.7$ kips	$\phi P_n = 334.5$ kips	0.31
19	Fuse yielding region dimension	$F2_{max} = 2$ in	$F2 = 1.9$ in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max} = 1.2$	$F6 / F2 = 1.1$	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn} = 102.7$ kips	$\phi R_n = 161.7$ kips	1.00
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn} = 102.7$ kips	$\phi R_n = 128$ kips	0.80
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
Controlling Demand-Capacity Ratio				1.00

BEAM LOCAL BUCKLING & LATERAL BRACING REPORT

HCAI CALCULATIONS

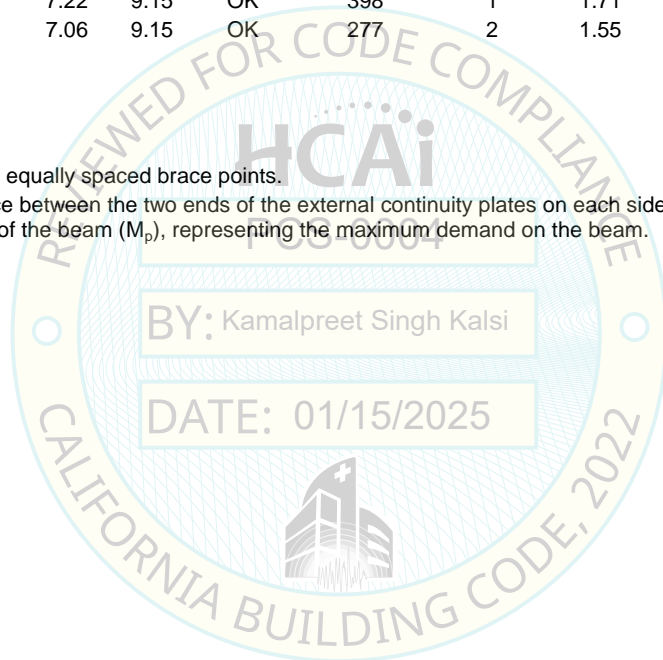
The purpose of this check is to demonstrate that the beams meet local buckling and bracing requirements specific to DuraFuse Frames, per ER 610. The beam is checked to ensure that it can reach the plastic moment prior to lateral-torsional buckling.

Beam	Col CL to CL Length (ft)	Bar to Bar Length (ft) ^b	λ	λ_p	λ Check	M_u (kip-ft) ^c	Lateral Brace Points Required	Lateral-Torsional Buckling Factor, C_b	Max Unbraced Length (ft)	ϕM_n - for LTB (kip-ft) ^a	$M_u/\phi M_n$ - for LTB (kip-ft)
W24X146	30.0	24.1	5.92	9.15	OK	1742	0	1.46	31.6	3079	0.57
W24X131	30.0	25.6	6.70	9.15	OK	1542	0	1.66	30.6	2566	0.60
W24X104	30.0	25.7	8.50	9.15	OK	1204	0	1.66	30.7	1901	0.63
W24X94	30.0	25.8	5.18	9.15	OK	1058	0	2.22	25.8	1099	0.96
W24X55	30.0	26.4	6.94	9.15	OK	558	1	1.80	13.5	582	0.96
W21X50	30.0	25.9	6.10	9.15	OK	458	1	1.81	13.0	478	0.96
W21X44	30.0	25.7	7.22	9.15	OK	398	1	1.71	12.5	401	0.99
W18X35	30.0	26.7	7.06	9.15	OK	277	2	1.55	10.0	293	0.94

a. Values reported correspond to equally spaced brace points.

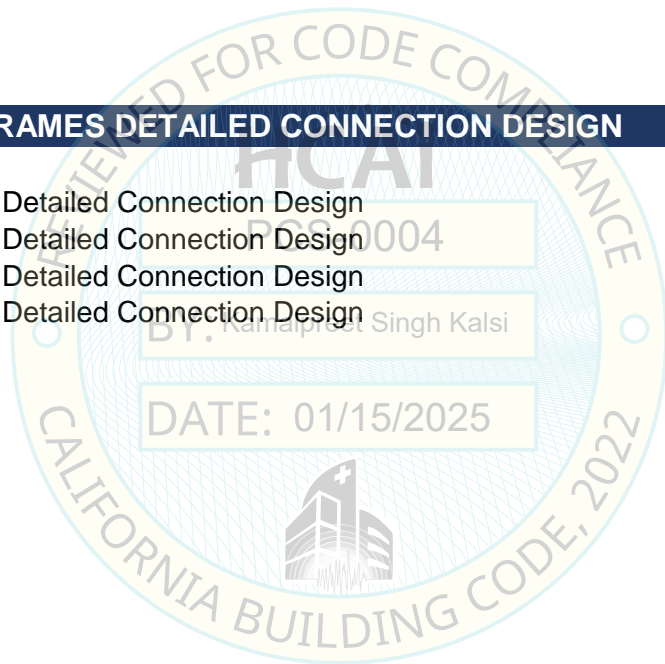
b. Bar to bar length is the distance between the two ends of the external continuity plates on each side of the beam.

c. M_u equals the plastic moment of the beam (M_p), representing the maximum demand on the beam.



3 DURAFUSE FRAMES DETAILED CONNECTION DESIGN

DF101 Detailed Connection Design	3.1
DF102 Detailed Connection Design	3.2
DF103 Detailed Connection Design	3.3
DF201 Detailed Connection Design	3.4



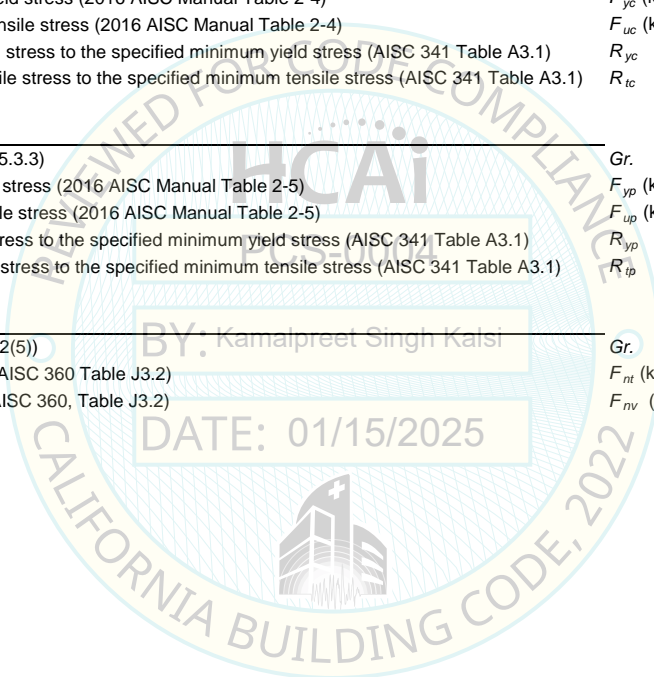
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF101
W21X122 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W24X146
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	24.7 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	146 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	33.2 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_yb)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	5.92 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))		See bracing calculations

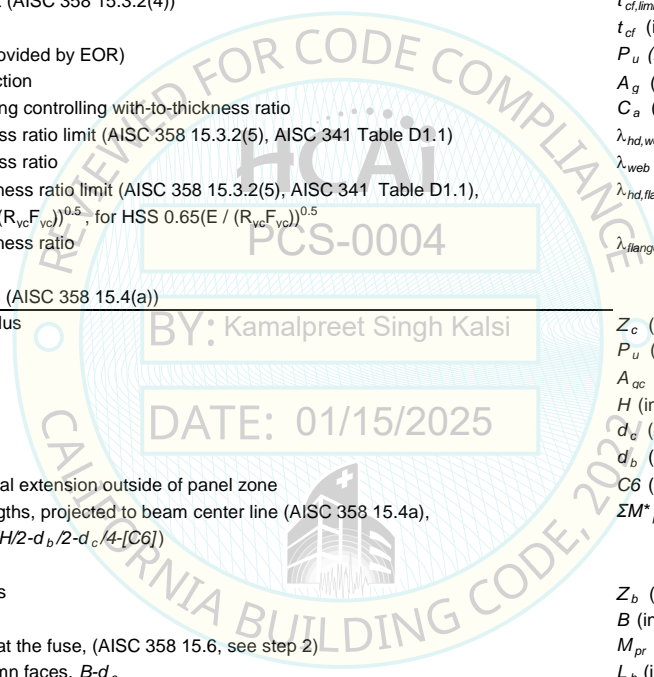
Column limitation checks

		W21X122
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	21.7
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	122 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.96 OK
Axial force on the column (provided by EOR)	P_u (kips)	345.3
Gross area of the column section	A_g (in ²)	35.9
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	50.2
Column web width-to-thickness ratio	λ_{web}	31.3 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{vc}F_{vc}))^{0.5}$; for HSS $0.65(E / (R_{vc}F_{vc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	6.45 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	307
Column design axial load	P_u (kips)	345.3
Column area	A_{gc} (in ²)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	3.07E+04
Beam plastic section modulus	Z_b (in ³)	418
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900
Clear distance between column faces, $B - d_c$	L_h (in)	338.3
Live load factor in seismic load combination	f_l	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f_lL + 0.2S$	V_g (kips)	43.8
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	167.4
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1816
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	2.27E+04
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	Ratio > 1.0	1.35 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1)



Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	6606
Beam plastic section modulus	Z_b (in ³)	418
Beam plastic bending moment, $F_{yb}Z_b$	M_p (k-in)	20900
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	20900 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	404.6
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	404.6

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Beam area	A_{ab} (in ²)	43.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	107.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	512.1
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	91
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	590.9
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhang dimension	C3 (in)	3.25
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	28.2
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	634.5
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	634.5
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.81 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.70
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	690.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	690.8
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.86 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.52
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.60 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bol} (in)	1.125
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Beam plastic section modulus	Z_b (in ³)	418
Beam flange thickness	t_{fb} (in)	1.09
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	1.499
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	70.8
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	347.2
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	107.5
Bolt diameter	d_{bol} (in)	1.125
Area of single bolt, $\pi*d_{bol}^2/4$	A_{bolt} (in ²)	0.994
Bolt shear strength, $\phi F_{nv} A_b$	ϕR_n (kips)	83.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi R_n$	$n_{b, req}$	6.1
Number of bolts per line (AISC 358 15.2(1))	n_b	9 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.375
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.125
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.843
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p, min}$	3.2
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	5 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	512.1
Beam flange thickness	t_{fb} (in)	1.09
Fuse and top plate thickness	$T2$ (in)	1.125
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	1.1
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bol}t_{min}F_{up}n_b$	R_{nb} (kips)	1721.7
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}t_{min}F_{up}(n_b - 1)$	R_{nt1} (kips)	1317.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o/2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}t_{min}F_{up}$	R_{nt2} (kips)	77.0
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1394.9
Nominal shear strength of all bolts per line, $F_{nv}A_{bolt}n_b$	R_{nv} (kips)	751.5
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	751.5
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	563.6
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91 OK

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1043
Bolt group demand from wind per line, $M_u/(2(d_b + t_p))$	R_u (kip)	20.2
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	9
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	244.1
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	207.5
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.10 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

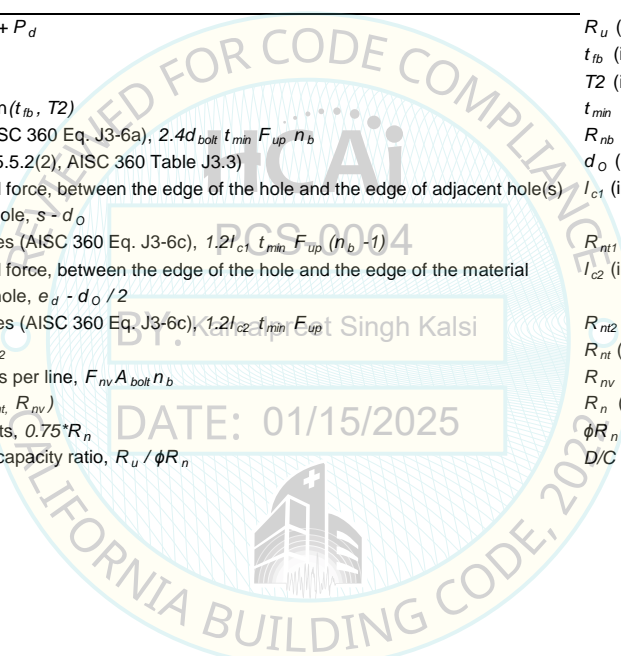
Maximum probable force per bolt line (see step 2)
 Total drag force through fuse plate (all drag force transmitted through top plate)
 Drag force per line of bolts, $P_{d,total}/2$
 Bolt diameter
 Area of single bolt, $\pi * d_{bolt}^2 / 4$
 Bolt shear strength, $\phi F_{nv} A_b$
 Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
 Number of bolts per line on bottom flange (AISC 358 15.2(1))
 Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
 Bolt spacing (see schedule)
 Plate thickness, t_p (see T2 in drawings)
 Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
 Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2[F2_a] + 3in - 2e_d] / s + 1$
 Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
 Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
 (AISC 358 Eq. 15.6-8), $n_b - n_p$

V_{fe} (kips)	404.6
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	1.125
A_{bolt} (in ²)	0.994
ϕr_n (kips)	83.5
$n_{b, req}$	4.8
$n_{b, Bot}$	7 OK
e_d (in)	1.625
s (in)	3.375
t_p (in)	1.125
$F2_a$ (in)	3.843
$n_{p,min}$	3.2
$n_{p, Bot}$	4 OK
$n_{m, Bot}$	3

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$
 Beam flange thickness
 Fuse and top plate thickness
 Governing material thickness, $\min(t_{fb}, T2)$
 Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
 Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
 Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 l_{c1} is based on an oversized hole, $s - d_o$
 Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$
 Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 l_{c2} is based on an oversized hole, $e_d - d_o / 2$
 Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$
 Total tearout strength, $R_{nt1} + R_{nt2}$
 Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
 Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
 Factored shear capacity of all bolts, $0.75 * R_n$
 Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	404.6
t_{fb} (in)	1.09
$T2$ (in)	1.125
t_{min} (in)	1.1
R_{nb} (kips)	1721.7
d_o (in)	1.4375
l_{c1} (in)	1.9
R_{nt1} (kips)	988.4
l_{c2} (in)	0.9
R_{nt2} (kips)	77.0
R_{nt} (kips)	1065.4
R_{nv} (kips)	584.5
R_n (kips)	584.5
ϕR_n (kips)	438.4
D/C	0.92 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.7
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	1
Cover plate overhang dimension	C3 (in)	3.25
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	C1 _{req} (in)	6.357
Distance from alignment line to column	C1 (in)	6.375 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	107.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	91
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.125
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.7
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	3.375
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.7500
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.20
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.88
Weld 1 size	D_{W1} (1/16 in)	10
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	0
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	3.75
Vertical distance from center of rotation to geometric center	e_y (in)	1.33

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) * L$ ΣRL (k-in) 17467

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R * L_R$ (k-in)
W1RT	17.17	15.45	65.57	265.3
W4TL	21.23	0.00	39.29	0.0
W1LB	19.53	13.70	41.64	267.7
W4BR	15.00	0.00	23.35	0.0
W1RC	7.28	9.34	12.85	68.0
W4TC	16.90	0.00	13.50	0.0
W1LC	14.69	10.15	6.33	149.1
W4BC	14.33	0.00	15.99	0.0
		15.45	65.57	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	15.4
Orientation of Resultant Force	θ_r (degrees)	65.6
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.58
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	17.9
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c * l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	17.9
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.22
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 * 0.6 * 70 * (1 + 0.5 * \sin^{1.5}(\theta_r)) * (p_i / (1.9 - 0.9p_i)))^{0.3} * 0.707 * D_{W1} / 16$	ϕr_n (kips)	19.84
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.90 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	C3 (in)	3.25
External continuity plate dimension	C5 (in)	3.375
Weld 2 size	D_{W2} (1/16 in)	10
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.63
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.34
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	18.2
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3] - 2l_{hb2} - l_{we2,eff}))$	r_{un2} (kip/in)	17.6
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	25.3
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1") (2 \text{ sides})$	ϕR_n (kip/in)	27.8
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.91 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1728.5
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	512.1
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp} (d_c + 2[C3])t_p$	ϕR_n (kips)	951.8
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	10064.8
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.71 OK

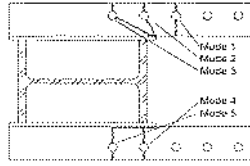
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column fillet dimension	k (in)	1.5
Column fillet dimension	$k1$ (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	91.5
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	10
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1697.4
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	809.2
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	2656.5
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1992.4
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.26 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	5.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	284.5
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	7.125
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Net area of external continuity plate cross section, $t_p((C4)-(d_{hole,std}+1/16))$	A_n (in ²)	6.539
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	425.0
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	318.8
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.89 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

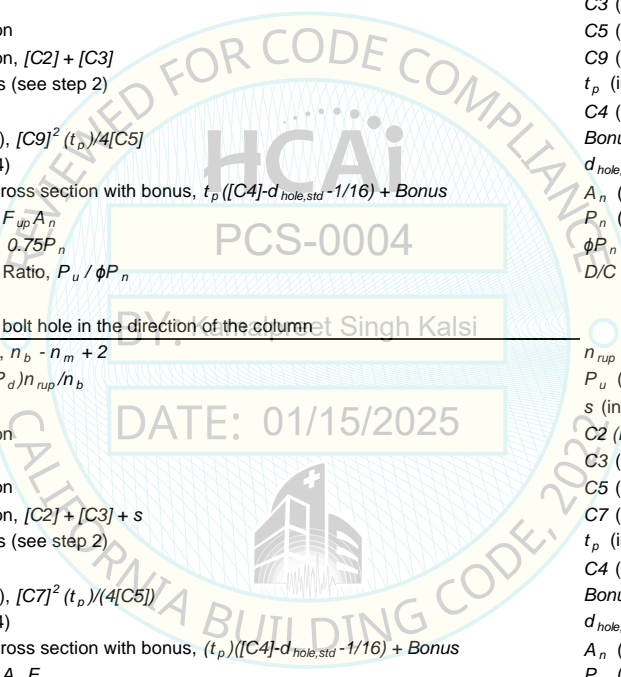
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	341.4
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate dimension	$C5$ (in)	3.375
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.375
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	7.125
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	$Bonus$ (in)	0.470
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $t_p((C4)-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	7.009
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	455.6
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	341.7
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	1.00 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	398.3
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate dimension	$C5$ (in)	3.375
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.75
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	7.125
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	$Bonus$ (in)	2.755
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $(t_p)((C4)-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	9.294
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	604.1
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	453.1
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.88 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	341.4
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate width	$C4$ (in)	7.125
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.375
External continuity plate thickness (see step 2)	t_p (in)	1.125
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	8.016
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	2.672
Net tension area, $A_{gt} - ((t_p)(d_{hole,std}+1/16))$	A_{nt} (in ²)	6.539
Net shear area, A_{gv}	A_{nv} (in ²)	2.672
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	505.2
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	378.9
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.90 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 5 rupture demand, $(V_{fe} + P_d) * n_{rup} / n_b$	R_u (kips)	398.3
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate width	$C4$ (in)	7.125
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.75
External continuity plate thickness (see step 2)	t_p (in)	1.125
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.25
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	8.016
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	6.469
Net tension area, $A_{gt} - (t_p)(d_{hole} + 1/16)$	A_{nt} (in ²)	6.539
Net shear area, A_{gv}	A_{nv} (in ²)	6.469
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	619.1
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	464.3
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.86 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	43.8
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	167.4
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	J_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	K_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.35 OK

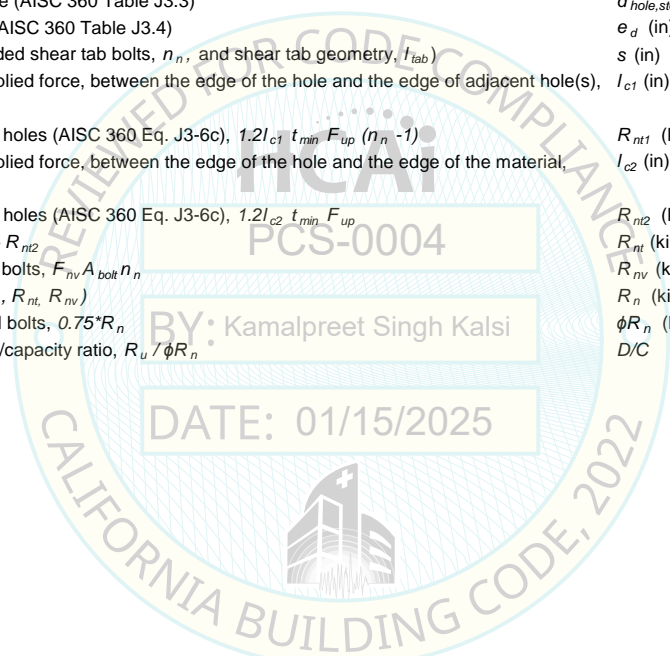
Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	107.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1024.3
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	9
Length of connection on beam	$B4$ (in)	28.625
Distance from beam centerline to bolt line	$P2$ (in)	2.75
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	8.066
Gross shear area, $2t_{fb} [B4]$	A_{gv} (in ²)	62.403
Net tension area, $A_{gt} - (t_{fb}(d_{hole, std} + 1/16))$	A_{nt} (in ²)	6.635
Net shear area, $A_{gv} - (t_{fb}(d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	38.082
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1916.5
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1437.4
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.71 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	167.4
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.994
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 OK
<hr/>		
Shear tab geometry requirements		
Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK
<hr/>		
Web bolt limit states check		
Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	$T3$ (in)	0.625
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.6
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	438.8
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	597.2
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	42.7
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	639.8
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.67 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	167.4
Distance from alignment line to column	$C1$ (in)	6.375
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1067.1

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bol} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.625
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	11.88
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	8.59
Effective net area, $A_e = A_{nv}$	A_e (in ²)	8.59
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	251.37
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	418.95
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.35 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	167.39
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1067.11
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	11.88
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	56.41
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	356.25
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{tn} (kips)	534.38
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2538.28
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.56 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.45
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	167.4
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	7.5
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	6.375
Size of weld in 1/16 th s of an inch	D_{w3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1067.1
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.2
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{Exx})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.93 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.7
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	512.1

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.125
Top plate dimension (see schedule)	$P6$ (in)	31
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	34.875
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1046.3
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1046.3
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.49 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	9
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	19.688
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	767.8
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	575.9
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89 OK

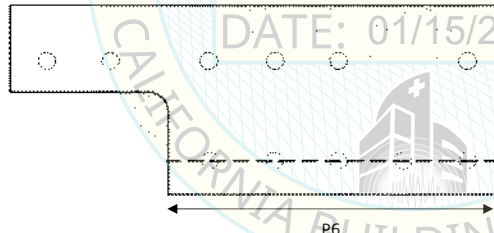


Fig. C-15.8. Shear rupture of net section on top plate.

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Total number of bolts (see step 5)	n_b	9
Number of bolts in the M region (see step 5)	n_m	4
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	227.6

Yielding check

Top plate dimension	$P4$ (in)	2.125
Top plate dimension	$P5$ (in)	3.625
Top plate thickness (see step 2)	t_p (in)	1.125
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	6.469
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	323.4
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	291.1
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.78 OK

Rupture check

Top plate dimension	$P4$ (in)	2.125
Top plate dimension	$P5$ (in)	3.625
Top plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	4.781
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	310.8
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	233.1
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 OK

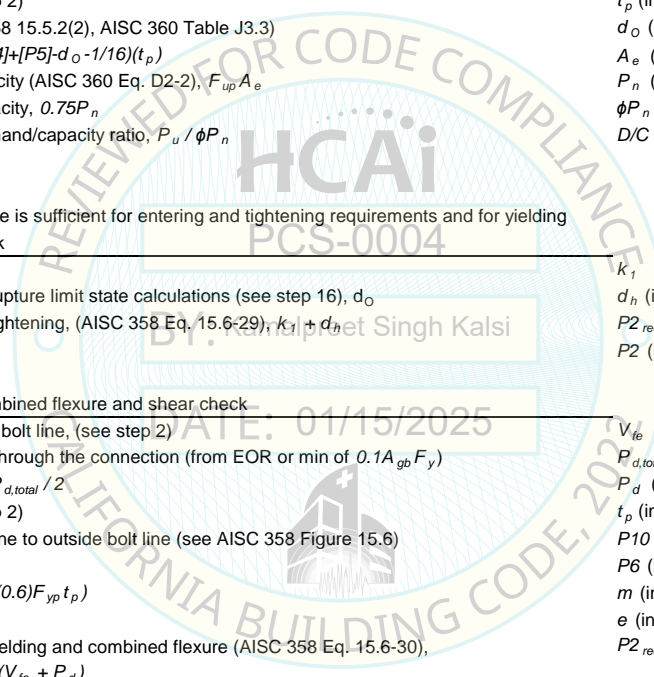
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.6
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3
Provided P2 dimension	$P2$ (in)	2.75 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Top plate thickness (see step 2)	t_p (in)	1.125
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.325
Inside length of the top plate	$P6$ (in)	31.00
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	16.861
Dimension, $([P6] - m) / 2$	e (in)	7.070
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-6.4
Provided P2 dimension	$P2$ (in)	2.75 OK



Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.75 OK
Top plate thickness (see step 2)	t_p (in)	1.125
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.4 OK

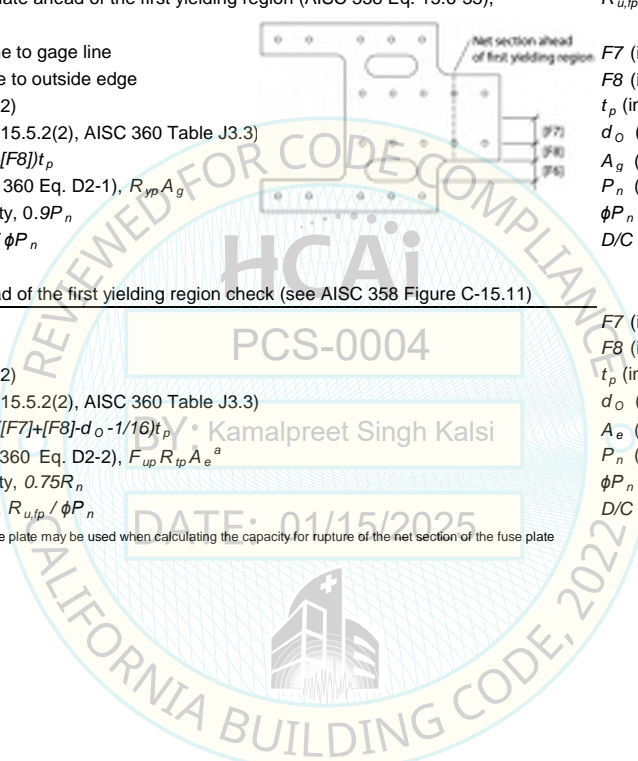
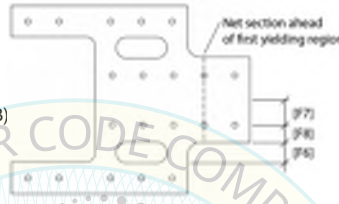
Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	7
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	3
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	346.8
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.7
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	12.263
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	613.1
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	551.8
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.63 OK

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.7
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	8.888
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_u R_{tp} A_e^a$	P_n (kips)	693.2
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	519.9
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.67 OK

^aThe expected tensile strength ($F_u R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Step 19. Determine the yielding region depth [F2]

Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	404.6
Fuse plate thickness(see step 2)	t_p (in)	1.125
Width of fuse plate cut-out	$F6$ (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{ip})B[F6])/(2t_p(0.6F_{up}R_{ip})[A-C[F6]/t_p])$	$F2_{max}$ (in)	3.147
Provided yielding region depth	$F2$ (in)	3.125 OK
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	210.9
Ratio	V_{fe} / V_y	1.9
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	0.88 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	7
Number of bolts in the M region	$n_{m, Bot}$	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	346.8
Yielding check		
Fuse plate dimension	$F4$ (in)	2.125
Fuse plate dimension	$F5$ (in)	2.125
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	9.5625
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	R_n (kips)	478.13
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	430.31
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	2.125
Fuse plate dimension	$F5$ (in)	2.125
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	6.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e^a$	R_n (kips)	482.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	362.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.96 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	107.5
External continuity plate dimension	C5 (in)	3.375
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	69.3
Cover plate overhang	C3 (in)	3.25
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	1.000
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕP_n (kips)	101.3
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	0.68 OK

Bridge plate to cover plate weld failure check

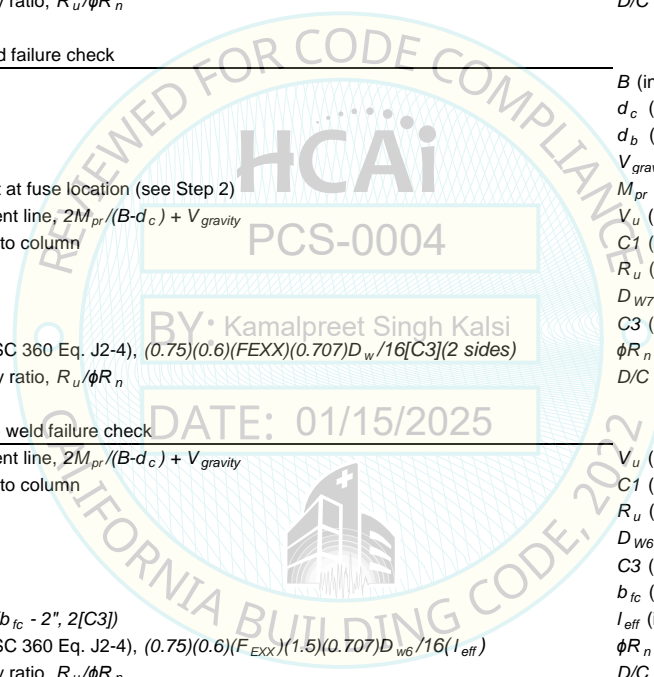
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	69.3
Weld 5 size	D_{W5} (1/16 in)	8
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16([C3]-1)(2 \text{ sides})$	ϕR_n (kips)	75.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.92 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Shear force from gravity	$V_{gravity}$ (kips)	43.8
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	167.4
Distance from alignment line to column	C1 (in)	6.375
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	43.2
Weld 7 size	D_{W7} (1/16 in)	5
Cover plate overhang	C3 (in)	3.25
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_w/16[C3](2 \text{ sides})$	ϕR_n (kips)	45.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.96 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	167.4
Distance from alignment line to column	C1 (in)	6.375
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	43.2
Weld 6 size	D_{W6} (1/16 in)	5
Cover plate overhang	C3 (in)	3.25
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	6.5
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	67.9
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.64 OK



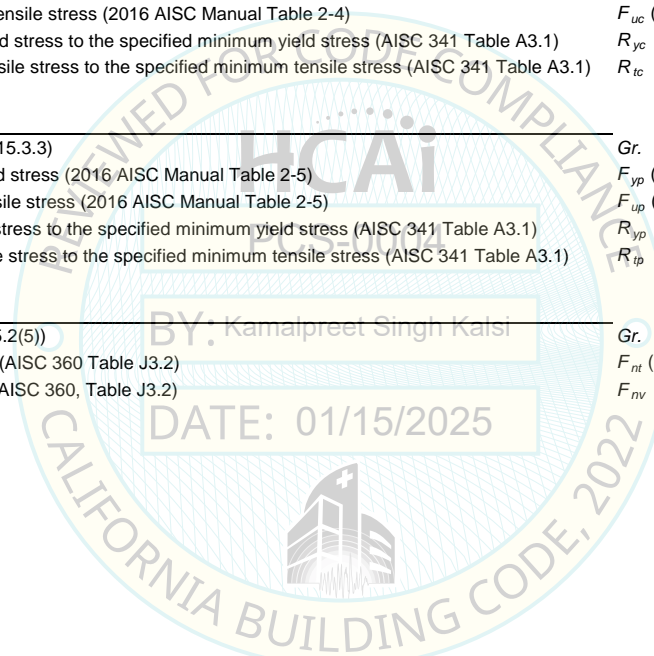
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF102
W21X73 Column and W24X94 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W24X94
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	24.3 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	94 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	41.9 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_yb)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	5.18 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))		See bracing calculations

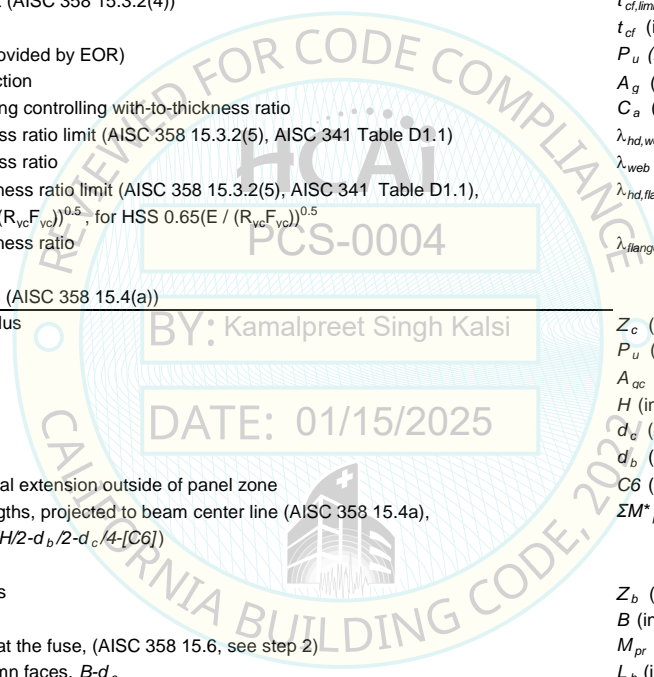
Column limitation checks

		W21X73
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	21.2
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	73 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.74 OK
Axial force on the column (provided by EOR)	P_u (kips)	212.5
Gross area of the column section	A_g (in ²)	21.5
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	50.1
Column web width-to-thickness ratio	λ_{web}	41.2 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{vc}F_{vc}))^{0.5}$; for HSS $0.65(E / (R_{vc}F_{vc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	5.6 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	172
Column design axial load	P_u (kips)	212.5
Column area	A_{gc} (in ²)	21.5
Story Height	H (in)	216.0
Column depth	d_c (in)	21.2
Beam depth	d_b (in)	24.3
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	1.70E+04
Beam plastic section modulus	Z_b (in ³)	254
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	12700
Clear distance between column faces, $B - d_c$	L_h (in)	338.8
Live load factor in seismic load combination	f_l	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f_lL + 0.2S$	V_g (kips)	58.1
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	133.1
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1410
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	1.41E+04
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	Ratio > 1.0	1.21 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1)



Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	5374
Beam plastic section modulus	Z_b (in ³)	254
Beam plastic bending moment, $F_{yb}Z_b$	M_p (k-in)	12700
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	12700 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1
Beam depth	d_b (in)	24.3
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	251.0
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	251.0

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.0
Beam area	A_{ob} (in ²)	27.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{ob}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	69.3
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	320.2
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	50
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1
Beam depth	d_b (in)	24.3
Column depth	d_c (in)	21.2
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	366.0
<hr/>		
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhang dimension	C3 (in)	3
Column depth	d_c (in)	21.2
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	27.2
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	612.0
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	612.0
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.52 OK
<hr/>		
Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.30
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	681.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	681.8
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.54 OK
<hr/>		
Cover plate thickness check (AISC 341 E3.6e.2)		
Beam depth	d_b (in)	24.3
Beam flange thickness	t_{fb} (in)	0.875
Column flange thickness	t_{fc} (in)	0.74
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.55
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.72
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.60 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bol} (in)	1.125
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Beam plastic section modulus	Z_b (in ³)	254
Beam flange thickness	t_{fb} (in)	0.875
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	1.203
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	56.4
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	197.6
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	13970
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	14131
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.99 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	138.5
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	69.3
Bolt diameter	d_{bol} (in)	1.125
Area of single bolt, $\pi*d_{bol}^2/4$	A_{bolt} (in ²)	0.994
Bolt shear strength, $\phi F_{nv} A_b$	ϕR_n (kips)	83.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi R_n$	$n_{b, req}$	3.8
Number of bolts per line (AISC 358 15.2(1))	n_b	6 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.375
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	2.681
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p, min}$	2.5
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	3

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	320.2
Beam flange thickness	t_{fb} (in)	0.875
Fuse and top plate thickness	$T2$ (in)	1
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	0.9
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bol}t_{min}F_{up}n_b$	R_{nb} (kips)	921.4
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}t_{min}F_{up}(n_b - 1)$	R_{nt1} (kips)	661.2
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o/2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}t_{min}F_{up}$	R_{nt2} (kips)	61.9
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	723.0
Nominal shear strength of all bolts per line, $F_{nv}A_{bolt}n_b$	R_{nv} (kips)	501.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	501.0
Factored shear capacity of all bolts, $0.75R_n$	ϕR_n (kips)	375.7
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.85 OK

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1915
Bolt group demand from wind per line, $M_u/(2(d_b + t_p))$	R_u (kip)	37.9
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	6
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	162.7
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	138.3
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.27 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

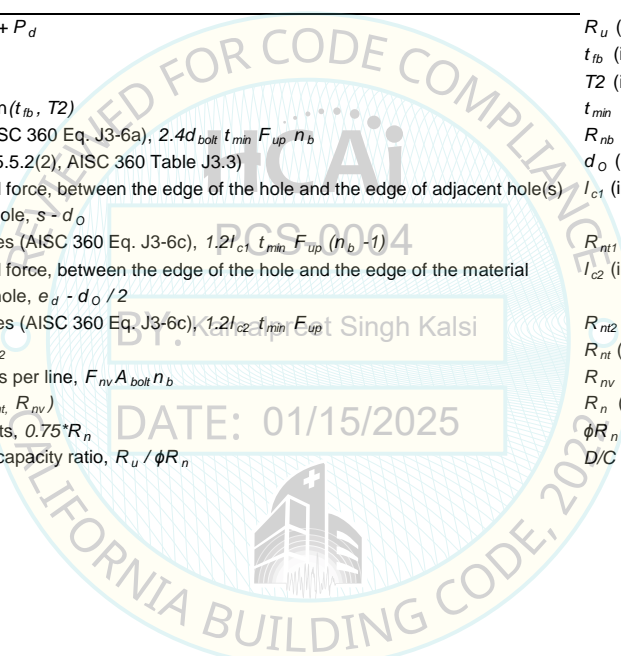
Maximum probable force per bolt line (see step 2)
 Total drag force through fuse plate (all drag force transmitted through top plate)
 Drag force per line of bolts, $P_{d,total}/2$
 Bolt diameter
 Area of single bolt, $\pi * d_{bolt}^2 / 4$
 Bolt shear strength, $\phi F_{nv} A_b$
 Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
 Number of bolts per line on bottom flange (AISC 358 15.2(1))
 Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
 Bolt spacing (see schedule)
 Plate thickness, t_p (see T2 in drawings)
 Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
 Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2[F2_a] + 3in - 2e_d] / s + 1$
 Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
 Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
 (AISC 358 Eq. 15.6-8), $n_b - n_p$

V_{fe} (kips)	251.0
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	1.125
A_{bolt} (in ²)	0.994
ϕr_n (kips)	83.5
$n_{b, req}$	3.0
$n_{b, Bot}$	5 OK
e_d (in)	1.625
s (in)	3.375
t_p (in)	1
$F2_a$ (in)	2.681
$n_{p,min}$	2.5
$n_{p, Bot}$	3 OK
$n_{m, Bot}$	2

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$
 Beam flange thickness
 Fuse and top plate thickness
 Governing material thickness, $\min(t_{fb}, T2)$
 Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
 Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
 Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 l_{c1} is based on an oversized hole, $s - d_o$
 Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$
 Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 l_{c2} is based on an oversized hole, $e_d - d_o / 2$
 Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$
 Total tearout strength, $R_{nt1} + R_{nt2}$
 Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
 Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
 Factored shear capacity of all bolts, $0.75 * R_n$
 Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	251.0
t_{fb} (in)	0.875
$T2$ (in)	1
t_{min} (in)	0.9
R_{nb} (kips)	921.4
d_o (in)	1.4375
l_{c1} (in)	1.9
R_{nt1} (kips)	528.9
l_{c2} (in)	0.9
R_{nt2} (kips)	61.9
R_{nt} (kips)	590.8
R_{nv} (kips)	417.5
R_n (kips)	417.5
ϕR_n (kips)	313.1
D/C	0.80 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.3
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	1
Cover plate overhang dimension	C3 (in)	3
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	C1 _{req} (in)	6.083
Distance from alignment line to column	C1 (in)	6.125 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per weld, $P_{d,total}/2$	P_d (kips)	69.25
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	50
Column depth	d_c (in)	21.2
Column flange thickness	t_{fc} (in)	0.74
Beam depth	d_b (in)	24.3
Plate thickness (see T2 on drawings)	t_p (in)	1
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.3
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	2.875
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.30
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.75
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	0
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.72
Horizontal distance from center of rotation to geometric center	e_x (in)	3.33
Vertical distance from center of rotation to geometric center	e_y (in)	1.41

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) * L$ ΣRL (k-in) 10392

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R * L_R$ (k-in)
W1RT	17.36	9.28	65.25	161.2
W4TL	20.85	0.00	38.55	0.0
W1LB	19.02	8.33	42.90	158.4
W4BR	14.90	0.00	25.14	0.0
W1RC	7.46	5.67	13.19	42.4
W4TC	16.69	0.00	12.21	0.0
W1LC	14.04	6.10	6.97	85.6
W4BC	13.94	0.00	14.66	0.0
		9.28	65.25	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	9.3
Orientation of Resultant Force	θ_r (degrees)	65.3
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	11.0
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c * l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	11.0
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.23
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 * 0.6 * 70 * (1 + 0.5 * \sin^{1.5}(\theta_r))) * (p_i * (1.9 - 0.9p_i))^{0.3} * 0.707 * D_{W1} / 16$	ϕr_n (kips)	11.88
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.93 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	69.25
Column depth	d_c (in)	21.2
Column flange thickness	t_{fc} (in)	0.74
Cover plate overhang dimension	C3 (in)	3
External continuity plate dimension	C5 (in)	2.875
Weld 2 size	D_{W2} (1/16 in)	6
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.38
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.12
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	11.8
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3] - 2l_{hb2} - l_{we2,eff}))$	r_{un2} (kip/in)	10.0
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	15.5
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1") (2 \text{ sides})$	ϕR_n (kip/in)	16.7
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.93 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.0
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	920.7
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	320.2
External continuity plate thickness (see step 2)	t_p (in)	1.000
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp} (d_c + 2[C3])t_p$	ϕR_n (kips)	816.0
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	8323.2
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.50 OK

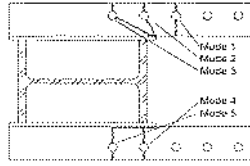
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	69.3
Column fillet dimension	k (in)	1.2
Column fillet dimension	$k1$ (in)	0.9
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	90.3
Column flange width	b_{fc} (in)	8.3
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	29.7
Weld 2 size	D_{W2} (1/16 in)	6
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1006.0
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	330.7
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	1351.1
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1013.4
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.32 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	69.3
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	160.1
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	$C4$ (in)	5.25
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Net area of external continuity plate cross section, $t_p((C4)-(d_{hole,std}+1/16))$	A_n (in ²)	3.938
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	255.9
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	192.0
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.83 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

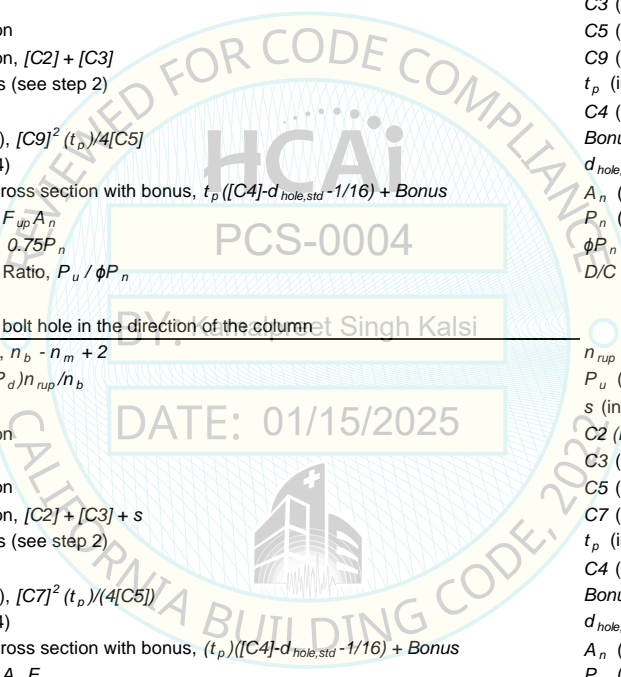
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	213.5
External continuity plate dimension	$C2$ (in)	-0.5
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate dimension	$C5$ (in)	2.875
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.5
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	$C4$ (in)	5.25
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	$Bonus$ (in)	0.543
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $t_p((C4)-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	4.481
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	291.3
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	218.4
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.98 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	266.9
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-0.5
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate dimension	$C5$ (in)	2.875
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.875
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	$C4$ (in)	5.25
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	$Bonus$ (in)	3.001
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $(t_p)((C4)-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	6.939
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	451.0
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	338.3
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.79 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	213.5
External continuity plate dimension	$C2$ (in)	-0.5
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate width	$C4$ (in)	5.25
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.5
External continuity plate thickness (see step 2)	t_p (in)	1
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	5.250
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	2.500
Net tension area, $A_{gt} - ((t_p)(d_{hole,std}+1/16))$	A_{nt} (in ²)	3.938
Net shear area, A_{gv}	A_{nv} (in ²)	2.500
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	330.9
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	248.2
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.86 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 5 rupture demand, $(V_{fe} + P_d) * n_{rup} / n_b$	R_u (kips)	266.9
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-0.5
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate width	$C4$ (in)	5.25
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.875
External continuity plate thickness (see step 2)	t_p (in)	1
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.25
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	5.250
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	5.875
Net tension area, $A_{gt} - (t_p)(d_{hole} + 1/16)$	A_{nt} (in ²)	3.938
Net shear area, A_{gv}	A_{nv} (in ²)	5.875
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	432.2
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	324.1
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.82 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	58.1
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	12700
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	133.1
Beam depth	d_b (in)	24.3
Beam web thickness	t_{wb} (in)	0.515
Beam web area, $d_b t_{wb}$	A_w (in)	12.5145
Beam web width-to-thickness ratio, h / t_w	J_{web}	41.9
Beam web shear buckling coefficient (AISC 360 G2.1)	K_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	375.4
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.35 OK

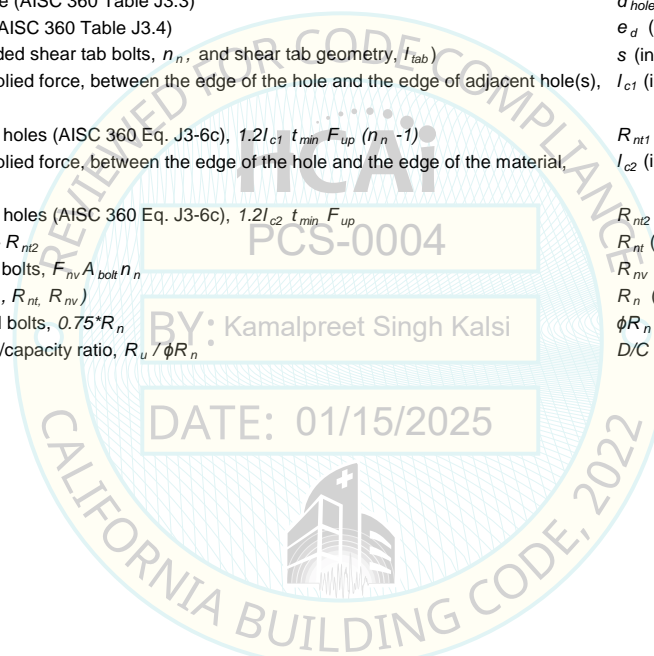
Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	138.5
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	69.3
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	640.5
Beam flange thickness	t_{fb} (in)	0.875
Beam flange width	b_{fb} (in)	9.07
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	6
Length of connection on beam	$B4$ (in)	18.5
Distance from beam centerline to bolt line	$P2$ (in)	2.75
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	3.124
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	32.375
Net tension area, $A_{gt} - (t_{fb}(d_{hole, std} + 1/16))$	A_{nt} (in ²)	1.975
Net shear area, $A_{gv} - (t_{fb}(d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	19.742
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	898.3
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	673.8
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.95 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	133.1
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.994
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	2
Provided number of shear tab bolts	n_n	4 OK
<hr/>		
Shear tab geometry requirements		
Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK
<hr/>		
Web bolt limit states check		
Beam web thickness	t_{wb} (in)	0.515
Shear tab thickness	$T3$ (in)	0.5
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	351.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	477.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	34.1
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	511.9
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.53 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	133.1
Distance from alignment line to column	$C1$ (in)	6.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	814.9

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bol} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	6.88
Effective net area, $A_e = A_{nv}$	A_e (in ²)	6.88
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	201.09
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	335.16
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u / \phi R_{tn})^2 + (V_u / \phi R_{nv})^4$	D/C	0.43 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	133.05
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	814.93
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{tn} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u / \phi M_n) + (P_u / \phi R_{tn})^2 + (V_u / \phi R_{nv})^4$	D/C	0.59 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.3
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	133.1
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	6.0
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	6.125
Size of weld in 1/16 th s of an inch	D_{w3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	814.9
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u) / l_{w3}^2 - 2F_b n_{vb} / l_{w3}$ except when bridge plates present, $F_b n_{vb} / l_{w3}$	r_{un} (k/in)	7.3
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	9.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.85 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.3
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.764
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	69.3
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	320.2

Yielding check

Top plate thickness (see step 2)	t_p (in)	1
Top plate dimension (see schedule)	$P6$ (in)	20.375
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	20.375
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	611.3
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	611.3
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.52 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	11.375
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	443.6
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	332.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.96 OK

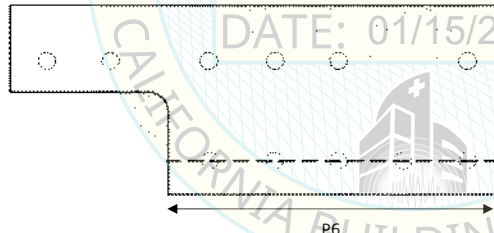


Fig. C-15.8. Shear rupture of net section on top plate.

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	69.3
Total number of bolts (see step 5)	n_b	6
Number of bolts in the M region (see step 5)	n_m	3
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	160.1

Yielding check

Top plate dimension	$P4$ (in)	1.875
Top plate dimension	$P5$ (in)	3
Top plate thickness (see step 2)	t_p (in)	1
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	4.875
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	243.8
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	219.4
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.73 OK

Rupture check

Top plate dimension	$P4$ (in)	1.875
Top plate dimension	$P5$ (in)	3
Top plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	3.375
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	219.4
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	164.5
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.97 OK

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.4
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.875
Provided P2 dimension	$P2$ (in)	2.75 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	69.3
Top plate thickness (see step 2)	t_p (in)	1
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	7.775
Inside length of the top plate	$P6$ (in)	20.38
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	11.861
Dimension, $([P6] - m) / 2$	e (in)	4.257
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-1.9
Provided P2 dimension	$P2$ (in)	2.75 OK

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	1.5 OK
Top plate thickness (see step 2)	t_p (in)	1
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	1.5 OK

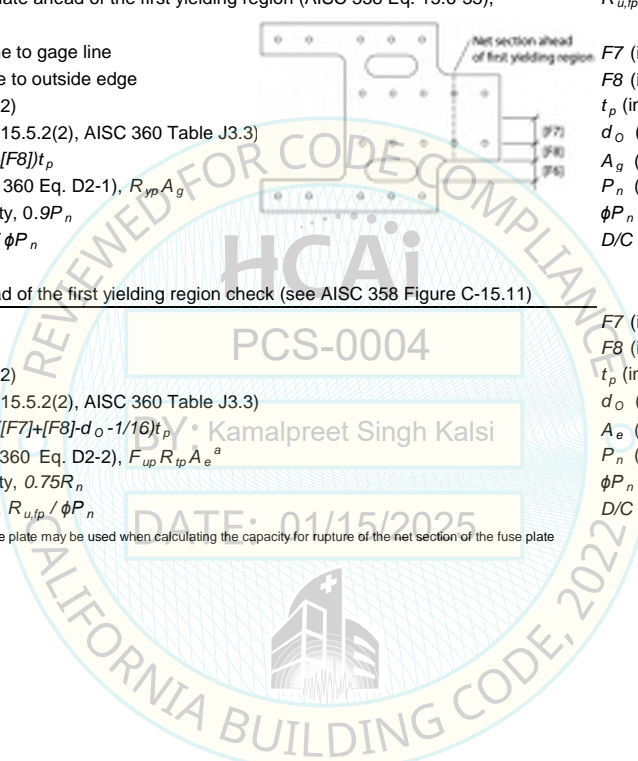
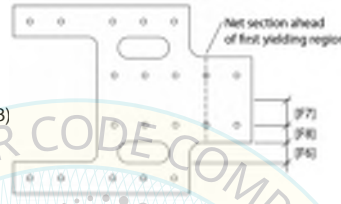
Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	5
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	2
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	200.8
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	1.65
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	8.800
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	440.0
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	396.0
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.51 OK

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	1.65
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	5.800
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_u R_{tp} A_e^a$	P_n (kips)	452.4
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	339.3
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.59 OK

^aThe expected tensile strength ($F_u R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Step 19. Determine the yielding region depth [F2]

Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	251.0
Fuse plate thickness(see step 2)	t_p (in)	1
Width of fuse plate cut-out	$F6$ (in)	1.5
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{ip})B[F6])/(2t_p(0.6F_{up}R_{ip})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.013
Provided yielding region depth	$F2$ (in)	2.000 OK
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	120.0
Ratio	V_{fe} / V_y	2.1
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	0.75 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	5
Number of bolts in the M region	$n_{m, Bot}$	2
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	200.8
Yielding check		
Fuse plate dimension	$F4$ (in)	1.875
Fuse plate dimension	$F5$ (in)	1.625
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	7
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	350.00
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	315.00
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	1.875
Fuse plate dimension	$F5$ (in)	1.625
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_o - 1/16)(t_p)$	A_e (in ²)	4
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	312.0
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	234.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.86 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_d / 2$	P_d (kips)	69.25
External continuity plate dimension	C5 (in)	2.875
Column depth	d_c (in)	21.2
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	38.0
Cover plate overhang	C3 (in)	3
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕP_n (kips)	56.3
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	0.68 OK

Bridge plate to cover plate weld failure check

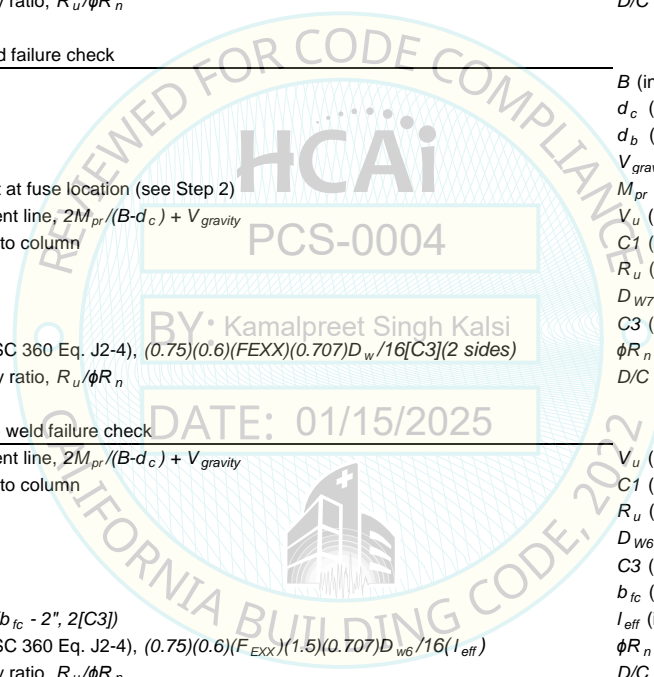
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	38.0
Weld 5 size	D_{W5} (1/16 in)	5
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16([C3]-1)(2 \text{ sides})$	ϕR_n (kips)	41.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.91 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	21.2
Beam depth	d_b (in)	24.3
Shear force from gravity	$V_{gravity}$ (kips)	58.1
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	12700
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	133.1
Distance from alignment line to column	C1 (in)	6.125
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	33.5
Weld 7 size	D_{W7} (1/16 in)	5
Cover plate overhang	C3 (in)	3
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_w/16[C3](2 \text{ sides})$	ϕR_n (kips)	41.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.80 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	133.1
Distance from alignment line to column	C1 (in)	6.125
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	33.5
Weld 6 size	D_{W6} (1/16 in)	4
Cover plate overhang	C3 (in)	3
Column flange width	b_{fc} (in)	8.3
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	6
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	50.1
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.67 OK



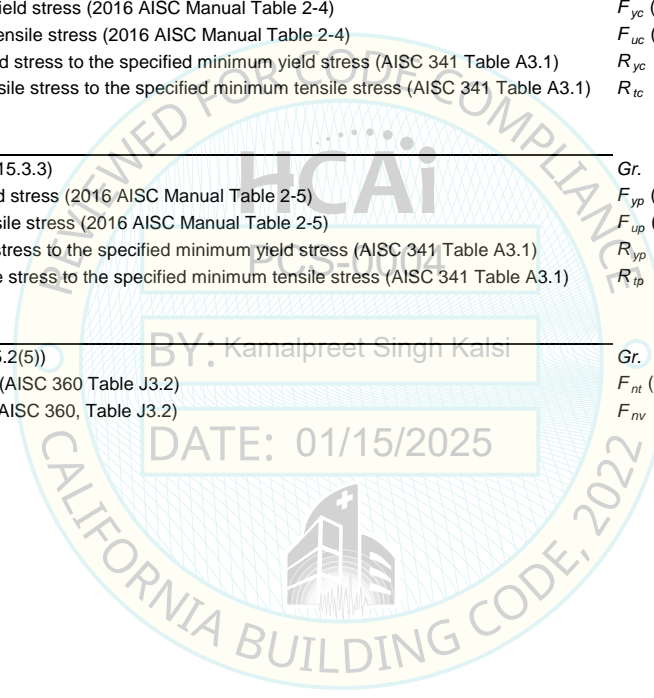
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF103
W16X57 Column and W24X55 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks		
Beam size		W24X55
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	23.6 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	55 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	54.6 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_yb)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	6.94 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))		See bracing calculations
Column limitation checks		
Column size		W16X57
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	16.4
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	57 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.715 OK
Axial force on the column (provided by EOR)	P_u (kips)	108.7
Gross area of the column section	A_g (in ²)	16.8
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.1
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	51.5
Column web width-to-thickness ratio	λ_{web}	33 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{vc}F_{vc}))^{0.5}$; for HSS $0.65(E / (R_{vc}F_{vc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	4.98 OK
Column-beam moment ratios (AISC 358 15.4(a))		
Column plastic section modulus	Z_c (in ³)	105
Column design axial load	P_u (kips)	108.7
Column area	A_{gc} (in ²)	16.8
Story Height	H (in)	216.0
Column depth	d_c (in)	16.4
Beam depth	d_b (in)	23.6
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	N/A
Beam plastic section modulus	Z_b (in ³)	134
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	5895
Clear distance between column faces, $B - d_c$	L_h (in)	343.6
Live load factor in seismic load combination	f_l	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f_lL + 0.2S$	V_g (kips)	31.5
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	65.9
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	540
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	N/A
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	Ratio > 1.0	N/A N/A

*Not applicable for top stories, see AISC 341 E3.4a(a1))

Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	828
Beam plastic section modulus	Z_b (in ³)	134
Beam plastic bending moment, $F_{yb}Z_b$	M_p (k-in)	6700
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	5895 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.625
Beam depth	d_b (in)	23.6
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	121.7
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	121.7

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	121.7
Beam area	A_{ab} (in ²)	16.2
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	81.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	40.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	162.2
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	32
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.625
Beam depth	d_b (in)	23.6
Column depth	d_c (in)	16.4
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	225.4
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhang dimension	C3 (in)	1
Column depth	d_c (in)	16.4
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	18.4
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	414.0
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	414.0
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.39 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag condition	h_{cp} (in)	25.98
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	584.4
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	584.4
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.39 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	23.6
Beam flange thickness	t_{fb} (in)	0.505
Column flange thickness	t_{fc} (in)	0.715
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.59
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	14.97
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.417
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.80 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bol} (in)	0.875
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	0.9375
Beam plastic section modulus	Z_b (in ³)	134
Beam flange thickness	t_{fb} (in)	0.505
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	0.537
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	24.8
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	109.2
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	7370
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	7809
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.94 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	81.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	40.5
Bolt diameter	d_{bol} (in)	0.875
Area of single bolt, $\pi*d_{bol}^2/4$	A_{bolt} (in ²)	0.601
Bolt shear strength, $\phi F_{nv} A_b$	ϕR_n (kips)	50.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi R_n$	$n_{b, req}$	3.2
Number of bolts per line (AISC 358 15.2(1))	n_b	5 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.1875
Bolt spacing (see schedule)	s (in)	3.875
Plate thickness, t_p (see T2 in drawings)	t_p (in)	0.625
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	2.080
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p, min}$	2.2
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	2
Flange bolt limit states check		
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	162.2
Beam flange thickness	t_{fb} (in)	0.505
Fuse and top plate thickness	$T2$ (in)	0.625
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bol}t_{min}F_{up}n_b$	R_{nb} (kips)	344.7
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.0625
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	2.8
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}t_{min}F_{up}(n_b - 1)$	R_{nt1} (kips)	443.1
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o/2$	l_{c2} (in)	0.7
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}t_{min}F_{up}$	R_{nt2} (kips)	25.8
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	469.0
Nominal shear strength of all bolts per line, $F_{nv}A_{bolt}n_b$	R_{nv} (kips)	252.6
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	252.6
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	189.4
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.86 OK
Bolt slip for wind load check		
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	148
Bolt group demand from wind per line, $M_u/(2(d_b + t_p))$	R_u (kip)	3.0
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	49
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	5
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	83.1
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	70.6
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.04 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

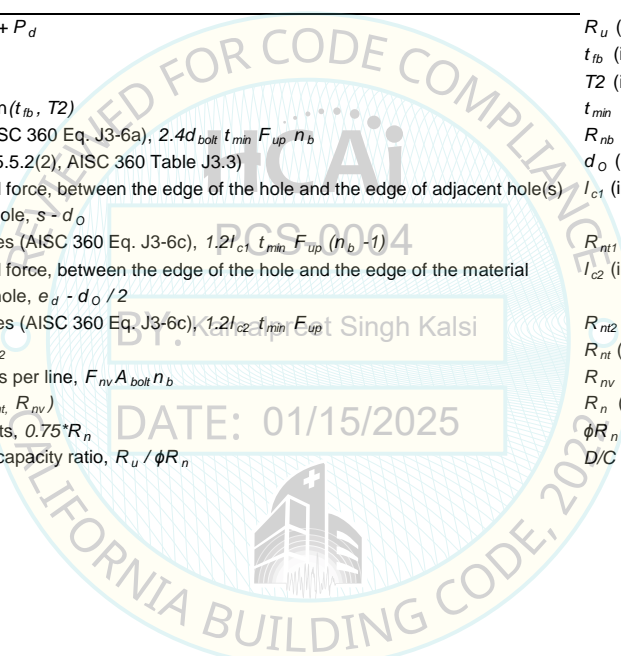
Maximum probable force per bolt line (see step 2)
 Total drag force through fuse plate (all drag force transmitted through top plate)
 Drag force per line of bolts, $P_{d,total}/2$
 Bolt diameter
 Area of single bolt, $\pi * d_{bolt}^2 / 4$
 Bolt shear strength, $\phi F_{nv} A_b$
 Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
 Number of bolts per line on bottom flange (AISC 358 15.2(1))
 Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
 Bolt spacing (see schedule)
 Plate thickness, t_p (see T2 in drawings)
 Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
 Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2[F2_a] + 3in - 2e_d] / s + 1$
 Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
 Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
 (AISC 358 Eq. 15.6-8), $n_b - n_p$

V_{fe} (kips)	121.7
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	0.875
A_{bolt} (in ²)	0.601
ϕr_n (kips)	50.5
$n_{b, req}$	2.4
$n_{b, Bot}$	5 OK
e_d (in)	1.1875
s (in)	3.875
t_p (in)	0.625
$F2_a$ (in)	2.080
$n_{p,min}$	2.2
$n_{p, Bot}$	3 OK
$n_{m, Bot}$	2

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$
 Beam flange thickness
 Fuse and top plate thickness
 Governing material thickness, $\min(t_{fb}, T2)$
 Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
 Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
 Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 l_{c1} is based on an oversized hole, $s - d_o$
 Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$
 Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 l_{c2} is based on an oversized hole, $e_d - d_o / 2$
 Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$
 Total tearout strength, $R_{nt1} + R_{nt2}$
 Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
 Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
 Factored shear capacity of all bolts, $0.75 * R_n$
 Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	121.7
t_{fb} (in)	0.505
$T2$ (in)	0.625
t_{min} (in)	0.5
R_{nb} (kips)	344.7
d_o (in)	1.0625
l_{c1} (in)	2.8
R_{nt1} (kips)	443.1
l_{c2} (in)	0.7
R_{nt2} (kips)	25.8
R_{nt} (kips)	469.0
R_{nv} (kips)	252.6
R_n (kips)	252.6
ϕR_n (kips)	189.4
D/C	0.64 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	23.6
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.1875
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	0
Cover plate overhang dimension	C3 (in)	1
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	C1 _{req} (in)	2.6035
Distance from alignment line to column	C1 (in)	4.25 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	81.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	40.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	32
Column depth	d_c (in)	16.4
Column flange thickness	t_{fc} (in)	0.715
Beam depth	d_b (in)	23.6
Plate thickness (see T2 on drawings)	t_p (in)	0.625
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	25.975
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	2.5
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.4375
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	25.10
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.38
Weld 1 size	D_{W1} (1/16 in)	5
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	0
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	14.97
Horizontal distance from center of rotation to geometric center	e_x (in)	2.12
Vertical distance from center of rotation to geometric center	e_y (in)	0.95

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) * L$ ΣRL (k-in) 6027

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R * L_R$ (k-in)
W1RT	14.58	6.62	65.36	96.5
W4TL	18.16	0.00	31.40	0.0
W1LB	15.34	6.12	47.71	93.9
W4BR	14.56	0.00	20.96	0.0
W1RC	6.19	3.96	111.18	24.5
W4TC	15.67	0.00	8.35	0.0
W1LC	10.39	4.23	6.63	44.0
W4BC	13.79	0.00	9.49	0.0
		6.62	65.36	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	6.6
Orientation of Resultant Force	θ_r (degrees)	65.4
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.50
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	7.5
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c * l_{We1})$	r_{un} (kip/in)	5.7
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	9.4
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.24
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 * 0.6 * 70 * (1 + 0.5 * \sin^{1.5}(\theta_r))) * (p_i * (1.9 - 0.9p_i))^{0.3} * 0.707 * D_{W1} / 16$	ϕr_n (kips)	9.89
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.95 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	81.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	40.5
Column depth	d_c (in)	16.4
Column flange thickness	t_{fc} (in)	0.715
Cover plate overhang dimension	C3 (in)	1
External continuity plate dimension	C5 (in)	2.5
Weld 2 size	D_{W2} (1/16 in)	6
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.38
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	2.09
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	8.8
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3] - 2l_{hb2} - l_{we2,eff}))$	r_{un2} (kip/in)	12.5
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	15.3
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1") (2 \text{ sides})$	ϕR_n (kip/in)	16.7
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.91 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	121.7
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	405.4
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	162.2
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp} (d_c + 2[C3])t_p$	ϕR_n (kips)	345.0
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	2380.5
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.64 OK

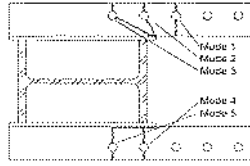
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	81.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	40.5
Column fillet dimension	k (in)	1.1
Column fillet dimension	$k1$ (in)	0.9
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	63.6
Column flange width	b_{fc} (in)	7.12
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	24.98
Weld 2 size	D_{W2} (1/16 in)	6
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	708.4
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	278.2
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	1019.4
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	764.5
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.21 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	81.0
Drag force per line of bolts, $P_d/total/2$	P_d (kips)	40.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	97.3
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate width	$C4$ (in)	4.25
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	0.9375
Net area of external continuity plate cross section, $t_p((C4)-(d_{hole,std}+1/16))$	A_n (in ²)	2.031
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	132.0
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	99.0
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

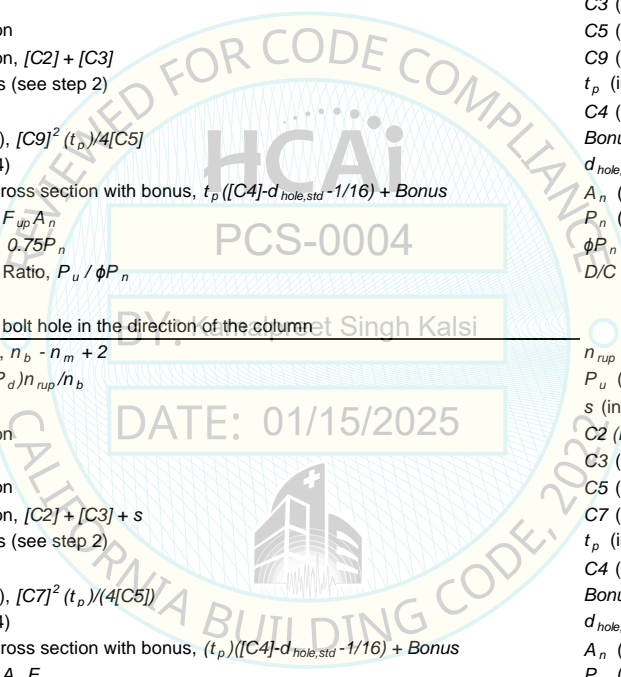
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	129.7
External continuity plate dimension	$C2$ (in)	2.75
Cover plate overhang dimension	$C3$ (in)	1
External continuity plate dimension	$C5$ (in)	2.5
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	3.75
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate width	$C4$ (in)	4.25
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	$Bonus$ (in)	0.879
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.9375
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	2.910
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	189.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	141.9
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.91 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	162.2
Bolt spacing (see schedule)	s (in)	3.875
External continuity plate dimension	$C2$ (in)	2.75
Cover plate overhang dimension	$C3$ (in)	1
External continuity plate dimension	$C5$ (in)	2.5
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	7.625
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate width	$C4$ (in)	4.25
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	$Bonus$ (in)	3.634
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.9375
Area of external continuity plate cross section with bonus, $(t_p)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	5.665
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	368.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	276.2
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.59 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	129.7
External continuity plate dimension	$C2$ (in)	2.75
Cover plate overhang dimension	$C3$ (in)	1
External continuity plate width	$C4$ (in)	4.25
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	3.75
External continuity plate thickness (see step 2)	t_p (in)	0.625
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.9375
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	2.656
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	2.344
Net tension area, $A_{gt} - ((t_p)(d_{hole,std}+1/16))$	A_{nt} (in ²)	2.031
Net shear area, A_{gv}	A_{nv} (in ²)	2.344
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	202.3
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	151.8
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.85 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 5 rupture demand, $(V_{fe} + P_d) * n_{rup} / n_b$	R_u (kips)	162.2
Bolt spacing (see schedule)	s (in)	3.875
External continuity plate dimension	$C2$ (in)	2.75
Cover plate overhang dimension	$C3$ (in)	1
External continuity plate width	$C4$ (in)	4.25
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	7.625
External continuity plate thickness (see step 2)	t_p (in)	0.625
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	0.9375
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	2.656
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	4.766
Net tension area, $A_{gt} - (t_p)(d_{hole} + 1/16)$	A_{nt} (in ²)	2.031
Net shear area, A_{gv}	A_{nv} (in ²)	4.766
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	275.0
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	206.3
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.79 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	31.5
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	5895
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	65.9
Beam depth	d_b (in)	23.6
Beam web thickness	t_{wb} (in)	0.395
Beam web area, $d_b t_{wb}$	A_w (in)	9.322
Beam web width-to-thickness ratio, h / t_w	J_{web}	54.6
Beam web shear buckling coefficient (AISC 360 G2.1)	K_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	279.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.24 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	81.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	40.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	324.4
Beam flange thickness	t_{fb} (in)	0.505
Beam flange width	b_{fb} (in)	7.01
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	0.9375
Number of holes along each bolt line (see step 5)	n_b	5
Length of connection on beam	$B4$ (in)	16.6875
Distance from beam centerline to bolt line	$P2$ (in)	2.375
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	1.141
Gross shear area, $2t_{fb} [B4]$	A_{gv} (in ²)	16.854
Net tension area, $A_{gt} - (t_{fb}(d_{hole, std} + 1/16))$	A_{nt} (in ²)	0.636
Net shear area, $A_{gv} - (t_{fb}(d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	12.309
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	521.4
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	391.1
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.83 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

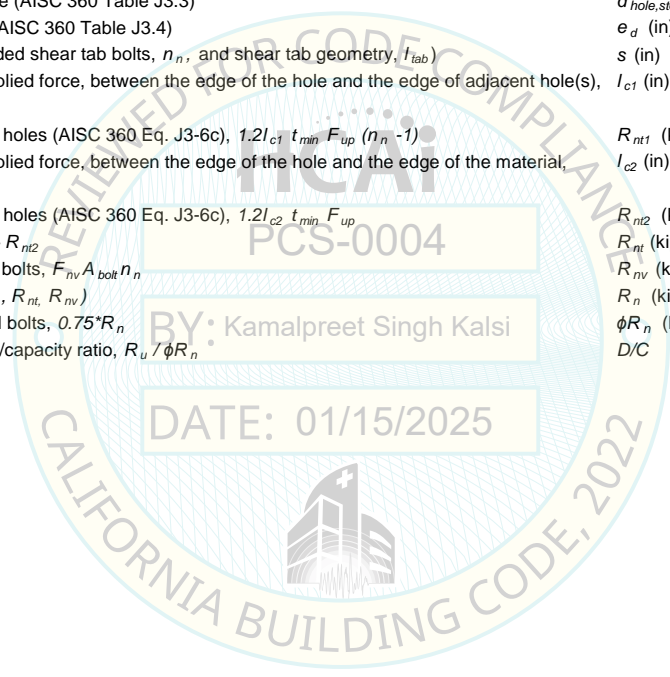
The required strength of shear tab bolt group (see step 10)	R_u (kips)	65.9
Bolt diameter (see step 4)	d_{bolt} (in)	0.875
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.601
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	50.5
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	37.9
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	2
Provided number of shear tab bolts	n_n	4 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20.75
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19.75
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19.75 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.395
Shear tab thickness	$T3$ (in)	0.5
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.4
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	215.7
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	0.9375
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.125
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.833
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	452.5
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.7
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	20.2
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	472.7
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	202.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	202.0
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	151.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.43 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	49
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	24.9
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	99.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	65.9
Distance from alignment line to column	$C1$ (in)	4.25
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	344.6

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bol} (in)	0.875
Slotted bolt hole width (standard slot width)	w_b (in)	0.9375
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19.75
Beam T dimension limit	T_{beam} (in)	20.75
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.125
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.75
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.88
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	7.88
Effective net area, $A_e = A_{nv}$	A_e (in ²)	7.88
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	230.34
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	383.91
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u / \phi R_{tn})^2 + (V_u / \phi R_{nv})^4$	D/C	0.07 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	65.86
Required axial strength of the shear tab (see above)	P_u (kips)	99.67
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	344.65
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.88
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	48.76
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	296.25
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{tn} (kips)	444.38
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2194.10
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u / \phi M_n) + (P_u / \phi R_{tn})^2 + (V_u / \phi R_{nv})^4$	D/C	0.21 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	19.75
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	65.9
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	3.3
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.75
Distance from line of bolts to shear tab weld	$C1$ (in)	4.25
Size of weld in 1/16 th s of an inch	D_{w3}	5
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	344.6
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u) / l_{w3}^2 - 2F_b n_{vb} / l_{w3}$ except when bridge plates present, $F_b n_{vb} / l_{w3}$	r_{un} (k/in)	12.8
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	13.3
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	13.9
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.95 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	0.875
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.1875
Beam depth	d_b (in)	23.6
Length of the shear tab (see step 12)	l_{tab} (in)	19.75
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.66675
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.75 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.75

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	81.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	40.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	162.2

Yielding check

Top plate thickness (see step 2)	t_p (in)	0.625
Top plate dimension (see schedule)	$P6$ (in)	19.25
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	12.031
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	360.9
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	360.9
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.45 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	0.875
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.125
Number of short slotted bolts on top plate (see step 5)	n_b	5
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	8.516
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	332.1
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	249.1
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.65 OK

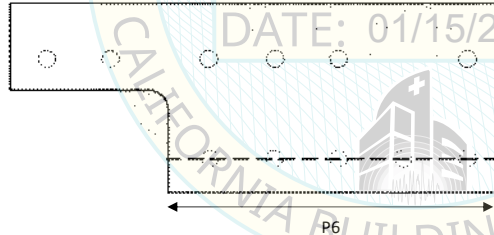


Fig. C-15.8. Shear rupture of net section on top plate.

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	81.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	40.5
Total number of bolts (see step 5)	n_b	5
Number of bolts in the M region (see step 5)	n_m	2
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	64.9

Yielding check

Top plate dimension	$P4$ (in)	1.625
Top plate dimension	$P5$ (in)	1.75
Top plate thickness (see step 2)	t_p (in)	0.625
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	2.109
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	105.5
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	94.9
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.68 OK

Rupture check

Top plate dimension	$P4$ (in)	1.625
Top plate dimension	$P5$ (in)	1.75
Top plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.0625
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	1.406
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	91.4
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	68.6
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.95 OK

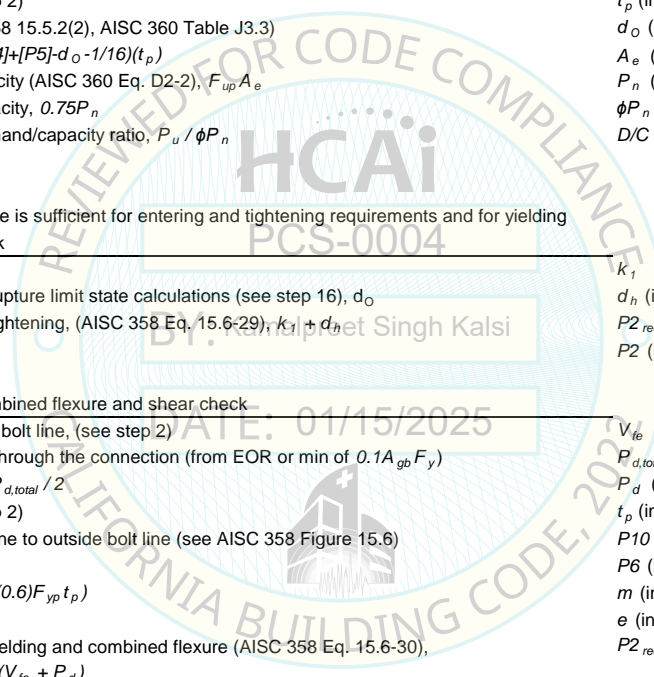
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.0
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.0625
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.0625
Provided P2 dimension	$P2$ (in)	2.375 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	81.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	40.5
Top plate thickness (see step 2)	t_p (in)	0.625
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	6.81
Inside length of the top plate	$P6$ (in)	19.25
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	9.611
Dimension, $([P6] - m) / 2$	e (in)	4.820
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-5.3
Provided P2 dimension	$P2$ (in)	2.375 OK



Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	1.5 OK
Top plate thickness (see step 2)	t_p (in)	0.625
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.4 OK

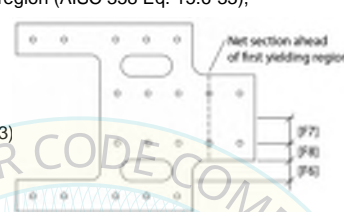
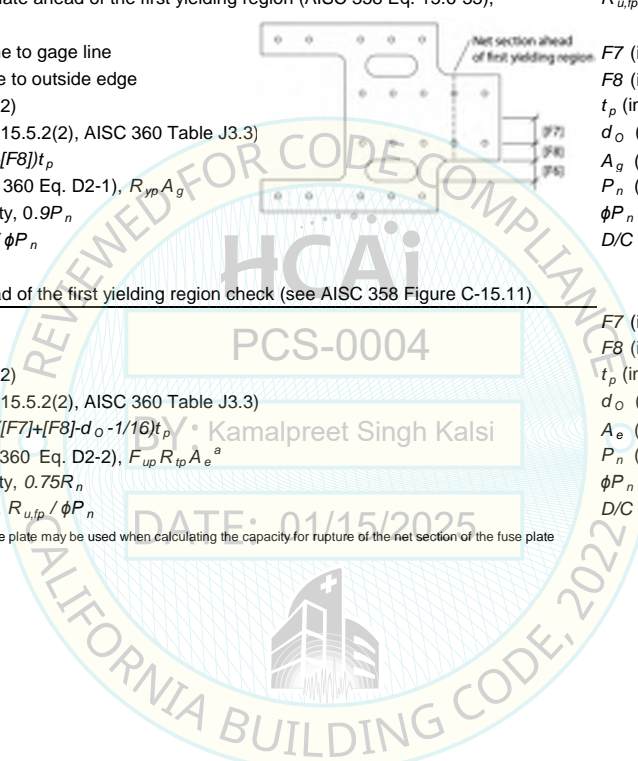
Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	5
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	2
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	97.3
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.375
Fuse plate dimension, gage line to outside edge	$F8$ (in)	1.31
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.0625
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	4.606
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	230.3
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	207.3
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.47 OK

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.375
Fuse plate dimension	$F8$ (in)	1.31
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.063
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	3.200
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_u R_{tp} A_e^a$	P_n (kips)	249.6
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	187.2
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.52 OK

^aThe expected tensile strength ($F_u R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Step 19. Determine the yielding region depth [F2]

Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	121.7
Fuse plate thickness(see step 2)	t_p (in)	0.625
Width of fuse plate cut-out	$F6$ (in)	1.5
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{ip})B[F6])/(2t_p(0.6F_{up}R_{ip})[A-C[F6]/t_p])$	$F2_{max}$ (in)	1.699
Provided yielding region depth	$F2$ (in)	1.625 OK
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	60.9
Ratio	V_{fe} / V_y	2.0
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	0.92 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

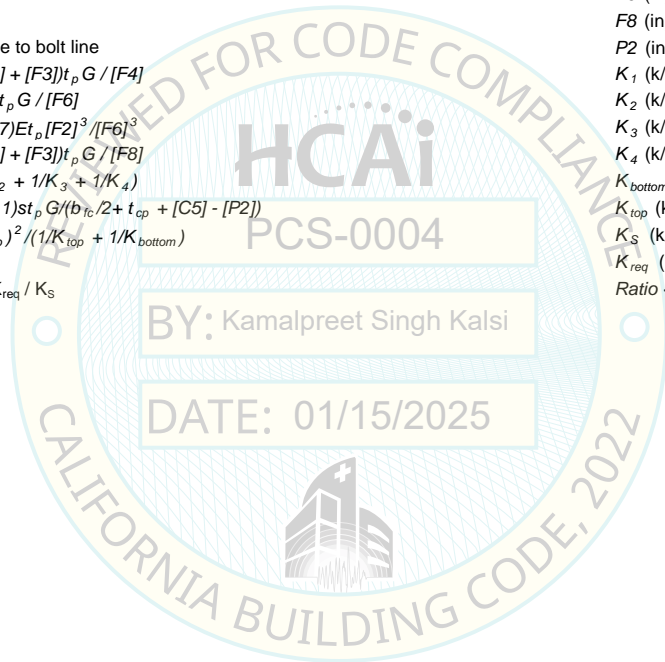
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	5
Number of bolts in the M region	$n_{m, Bot}$	2
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	97.3
Yielding check		
Fuse plate dimension	$F4$ (in)	1.625
Fuse plate dimension	$F5$ (in)	1.25
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.0625
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	3.59375
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	179.69
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	161.72
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	1.625
Fuse plate dimension	$F5$ (in)	1.25
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.0625
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	2.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	170.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	128.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.76 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Connection stiffness check

The purpose of this check is to ensure that the connection has enough stiffness to be considered Fully Restrained (FR) when M_{pr} is less than M_p (see AISC 358-22 15.6 Step 2). Moment frame connections are considered FR for design purposes if the connection stiffness is large relative to the beam flexural stiffness (EI/L). A threshold of $18EI/L$ is used in Chapter 13 of AISC 358 based on experimental testing. This procedure follows the method in the commentary to AISC 358-22 15.6 Step 2.

Beam moment of inertia	I_{xb} (in ³)	1350
Column flange width	b_{fc} (in)	7.12
Beam depth	d_b (in)	23.6
Bay width	B (in)	360.0
Number of bolts in zone M	M	2
Number of bolts in zone P	P	3
Bolt spacing	s (in)	3.875
Cover plate thickness	t_{cp} (in)	0.75
Fuse plate thickness	t_p (in)	0.625
External continuity plate dimension	$C5$ (in)	2.5
Fuse plate dimension	$F2$ (in)	1.625
Fuse plate dimension	$F3$ (in)	7.625
Fuse plate dimension	$F4$ (in)	1.6
Fuse plate dimension	$F6$ (in)	1.5
Fuse plate dimension	$F8$ (in)	1.31
Distance from beam centerline to bolt line	$P2$ (in)	2.375
Fuse region 1 stiffness, $(2[F2] + [F3])t_p G / [F4]$	K_1 (k/in)	4.68E+04
Fuse region 2 stiffness, $2[F2]t_p G / [F6]$	K_2 (k/in)	1.52E+04
Fuse region 3 stiffness, $2(0.37)Et_p [F2]^3 / [F6]^3$	K_3 (k/in)	1.71E+04
Fuse region 4 stiffness, $(2[F2] + [F3])t_p G / [F8]$	K_4 (k/in)	5.81E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	1.23E+04
Top plate stiffness, $(2P + M - 1)st_p G / (b_{fc}/2 + t_{cp} + [C5] - [P2])$	K_{top} (k/in)	4.28E+04
Connection stiffness, $(d_b + t_p)^2 / (1/K_{top} + 1/K_{bottom})$	K_S (k/in)	5.59E+06
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	1.96E+06
Connection stiffness check, K_{req} / K_S	$Ratio < 1$	0.35 OK



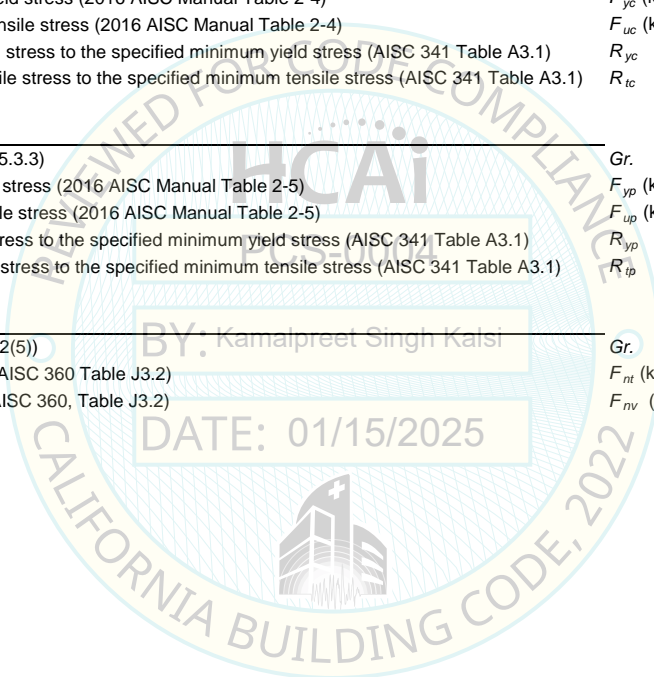
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF201
W21X166 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks		
Beam size		W24X146
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	24.7 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	146 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	33.2 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_yb)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	5.92 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))		See bracing calculations
Column limitation checks		
Column size		W21X166
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	22.5
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	166 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	1.36 OK
Axial force on the column (provided by EOR)	P_u (kips)	224.7
Gross area of the column section	A_g (in ²)	48.8
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.1
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	53.3
Column web width-to-thickness ratio	λ_{web}	25 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{vc}F_{vc}))^{0.5}$; for HSS $0.65(E / (R_{vc}F_{vc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	4.57 OK
Column-beam moment ratios (AISC 358 15.4(a))		
Column plastic section modulus	Z_c (in ³)	432
Column design axial load	P_u (kips)	224.7
Column area	A_{gc} (in ²)	48.8
Story Height	H (in)	216.0
Column depth	d_c (in)	22.5
Beam depth	d_b (in)	24.7
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	4.87E+04
Beam plastic section modulus	Z_b (in ³)	418
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900
Clear distance between column faces, $B - d_c$	L_h (in)	337.5
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	40.1
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	163.9
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1844
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	4.55E+04
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	Ratio > 1.0	1.07 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1))

Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	7221
Beam plastic section modulus	Z_b (in ³)	418
Beam plastic bending moment, $F_{yb}Z_b$	M_p (k-in)	20900
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	20900 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25
Beam depth	d_b (in)	24.7
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	402.7
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	402.7
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	805.4

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	805.4
Beam area	A_{ob} (in ²)	43.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{ob}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	107.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	912.9
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	89
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25
Beam depth	d_b (in)	24.7
Column depth	d_c (in)	22.5
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	1035.6
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	1.125
Cover plate overhang dimension	C3 (in)	6
Column depth	d_c (in)	22.5
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	34.5
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	1164.4
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	1164.4
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.78 OK
Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.70
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	1036.1
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	1036.1
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	1.00 OK
Cover plate thickness check (AISC 341 E3.6e.2)		
Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column flange thickness	t_{fc} (in)	1.36
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.52
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	2.39 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bol} (in)	1.125
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Beam plastic section modulus	Z_b (in ³)	418
Beam flange thickness	t_{fb} (in)	1.09
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	1.499
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	70.8
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	347.2
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	107.5
Bolt diameter	d_{bol} (in)	1.125
Area of single bolt, $\pi*d_{bol}^2/4$	A_{bolt} (in ²)	0.994
Bolt shear strength, $\phi F_{nv} A_b$	ϕR_n (kips)	83.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi R_n$	$n_{b, req}$	6.1
Number of bolts per line (AISC 358 15.2(1))	n_b	9 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.375
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.25
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.442
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p, min}$	3.0
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	5 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	510.2
Beam flange thickness	t_{fb} (in)	1.09
Fuse and top plate thickness	$T2$ (in)	1.25
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	1.1
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bol}t_{min}F_{up}n_b$	R_{nb} (kips)	1721.7
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}t_{min}F_{up}(n_b - 1)$	R_{nt1} (kips)	1317.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o/2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}t_{min}F_{up}$	R_{nt2} (kips)	77.0
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1394.9
Nominal shear strength of all bolts per line, $F_{nv}A_{bolt}n_b$	R_{nv} (kips)	751.5
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	751.5
Factored shear capacity of all bolts, $0.75R_n$	ϕR_n (kips)	563.6
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91 OK

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1926
Bolt group demand from wind per line, $M_u/(2(d_b + t_p))$	R_u (kip)	37.1
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	9
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	244.1
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	207.5
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.18 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

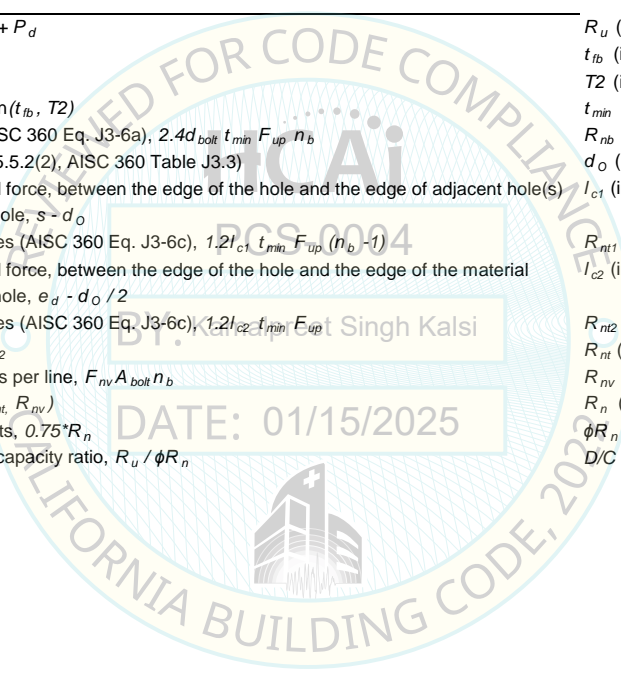
Maximum probable force per bolt line (see step 2)
 Total drag force through fuse plate (all drag force transmitted through top plate)
 Drag force per line of bolts, $P_{d,total}/2$
 Bolt diameter
 Area of single bolt, $\pi * d_{bolt}^2 / 4$
 Bolt shear strength, $\phi F_{nv} A_b$
 Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
 Number of bolts per line on bottom flange (AISC 358 15.2(1))
 Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
 Bolt spacing (see schedule)
 Plate thickness, t_p (see T2 in drawings)
 Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
 Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2(F2_a) + 3in - 2e_d] / s + 1$
 Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
 Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
 (AISC 358 Eq. 15.6-8), $n_b - n_p$

V_{fe} (kips)	402.7
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	1.125
A_{bolt} (in ²)	0.994
ϕr_n (kips)	83.5
$n_{b, req}$	4.8
$n_{b, Bot}$	7 OK
e_d (in)	1.625
s (in)	3.375
t_p (in)	1.25
$F2_a$ (in)	3.442
$n_{p,min}$	3.0
$n_{p, Bot}$	4 OK
$n_{m, Bot}$	3

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$
 Beam flange thickness
 Fuse and top plate thickness
 Governing material thickness, $\min(t_{fb}, T2)$
 Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
 Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
 Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 l_{c1} is based on an oversized hole, $s - d_o$
 Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$
 Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 l_{c2} is based on an oversized hole, $e_d - d_o / 2$
 Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$
 Total tearout strength, $R_{nt1} + R_{nt2}$
 Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
 Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
 Factored shear capacity of all bolts, $0.75 * R_n$
 Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	402.7
t_{fb} (in)	1.09
$T2$ (in)	1.25
t_{min} (in)	1.1
R_{nb} (kips)	1721.7
d_o (in)	1.4375
l_{c1} (in)	1.9
R_{nt1} (kips)	988.4
l_{c2} (in)	0.9
R_{nt2} (kips)	77.0
R_{nt} (kips)	1065.4
R_{nv} (kips)	584.5
R_n (kips)	584.5
ϕR_n (kips)	438.4
D/C	0.92 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.7
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	1
Cover plate overhang dimension	C3 (in)	6
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	C1 _{req} (in)	9.107
Distance from alignment line to column	C1 (in)	9.125 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	805.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	107.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	89
Column depth	d_c (in)	22.5
Column flange thickness	t_{fc} (in)	1.36
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.25
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	1.125
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.7
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	3.25
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.8125
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.08
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	5.38
Weld 1 size	D_{W1} (1/16 in)	11
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	9
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	1.36
Vertical distance from center of rotation to geometric center	e_y (in)	1.21

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) * L$ ΣRL (k-in) 30817

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R * L_R$ (k-in)
W1RT	18.35	16.25	57.40	298.1
W4TL	19.70	7.68	34.13	151.4
W1LB	18.14	15.43	45.94	280.0
W4BR	16.19	7.82	30.95	126.6
W1RC	10.00	10.58	8.64	105.8
W4TC	16.39	6.50	5.47	106.5
W1LC	12.70	10.91	6.79	138.6
W4BC	13.98	6.40	6.42	89.5
		16.25	57.40	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	16.2
Orientation of Resultant Force	θ_r (degrees)	57.4
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.58
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	20.2
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c * l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	20.2
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.28
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 * 0.6 * 70 * (1 + 0.5 * \sin^{1.5}(\theta_r))) * (p_i * (1.9 - 0.9p_i))^{0.3} * 0.707 * D_{W1} / 16$	ϕr_n (kips)	20.96
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.96 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	805.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column depth	d_c (in)	22.5
Column flange thickness	t_{fc} (in)	1.36
Cover plate overhang dimension	C3 (in)	6
External continuity plate dimension	C5 (in)	3.25
Weld 2 size	D_{W2} (1/16 in)	12
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.75
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	7.74
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	26.5
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3] - 2l_{hb2} - l_{we2,eff}))$	r_{un2} (kip/in)	15.2
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	30.5
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1") (2 \text{ sides})$	ϕR_n (kip/in)	33.4
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.91 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	805.4
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	2966.9
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	912.9
External continuity plate thickness (see step 2)	t_p (in)	1.250
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp} (d_c + 2[C3])t_p$	ϕR_n (kips)	1293.8
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	16737.9
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.88 OK

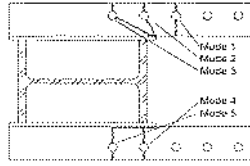
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	805.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column fillet dimension	k (in)	1.9
Column fillet dimension	$k1$ (in)	1.6
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	103.6
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	43.35
Weld 2 size	D_{W2} (1/16 in)	12
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	2306.3
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	965.4
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	3408.5
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	2556.4
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.36 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	5.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	283.4
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	$C4$ (in)	6.25
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Net area of external continuity plate cross section, $t_p((C4)-(d_{hole,std}+1/16))$	A_n (in ²)	6.172
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	401.2
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	300.9
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.94 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

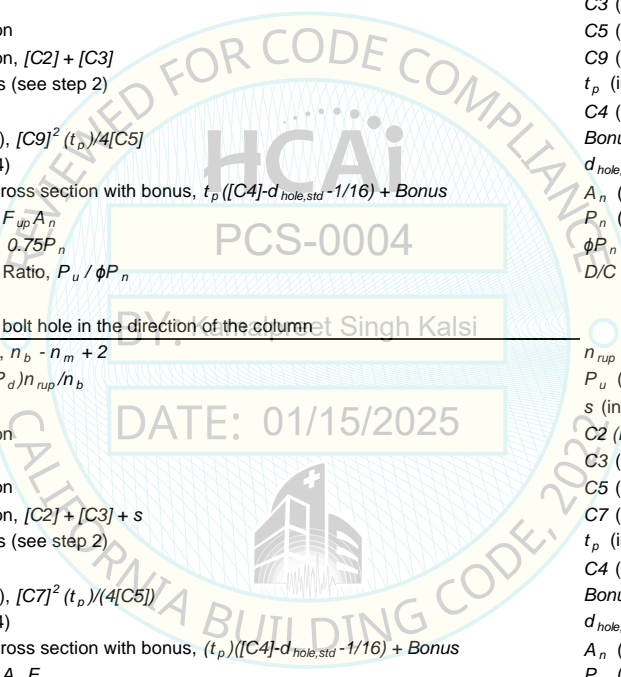
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	340.1
External continuity plate dimension	$C2$ (in)	-2.875
Cover plate overhang dimension	$C3$ (in)	6
External continuity plate dimension	$C5$ (in)	3.25
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	3.125
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	$C4$ (in)	6.25
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	$Bonus$ (in)	0.939
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $t_p((C4)-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	7.111
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	462.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	346.7
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.98 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	396.8
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-2.875
Cover plate overhang dimension	$C3$ (in)	6
External continuity plate dimension	$C5$ (in)	3.25
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	6.5
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	$C4$ (in)	6.25
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	$Bonus$ (in)	4.063
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $(t_p)((C4)-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	10.234
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	665.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	498.9
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.80 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	340.1
External continuity plate dimension	$C2$ (in)	-2.875
Cover plate overhang dimension	$C3$ (in)	6
External continuity plate width	$C4$ (in)	6.25
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	3.125
External continuity plate thickness (see step 2)	t_p (in)	1.25
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	7.813
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	3.906
Net tension area, $A_{gt} - ((t_p)(d_{hole,std}+1/16))$	A_{nt} (in ²)	6.172
Net shear area, A_{gv}	A_{nv} (in ²)	3.906
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	518.4
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	388.8
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.87 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 5 rupture demand, $(V_{fe} + P_d) * n_{rup} / n_b$	R_u (kips)	396.8
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-2.875
Cover plate overhang dimension	$C3$ (in)	6
External continuity plate width	$C4$ (in)	6.25
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	6.5
External continuity plate thickness (see step 2)	t_p (in)	1.25
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.25
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	7.813
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	8.125
Net tension area, $A_{gt} - (t_p)(d_{hole} + 1/16)$	A_{nt} (in ²)	6.172
Net shear area, A_{gv}	A_{nv} (in ²)	8.125
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	644.9
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	483.7
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.82 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	40.1
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	163.9
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	J_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	K_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.34 OK

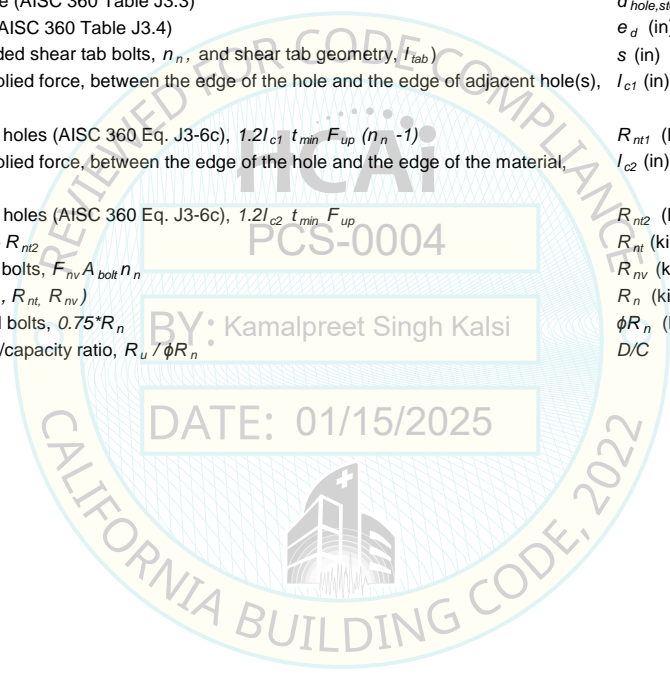
Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	107.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1020.4
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	9
Length of connection on beam	$B4$ (in)	28.625
Distance from beam centerline to bolt line	$P2$ (in)	2.75
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	8.066
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	62.403
Net tension area, $A_{gt} - (t_{fb}(d_{hole, std} + 1/16))$	A_{nt} (in ²)	6.635
Net shear area, $A_{gv} - (t_{fb}(d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	38.082
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1916.5
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1437.4
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.71 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	163.9
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.994
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 OK
<hr/>		
Shear tab geometry requirements		
Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK
<hr/>		
Web bolt limit states check		
Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	$T3$ (in)	0.625
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.6
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	438.8
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	597.2
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	42.7
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	639.8
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.65 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	163.9
Distance from alignment line to column	$C1$ (in)	9.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1495.8

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bol} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.625
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	11.88
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	8.59
Effective net area, $A_e = A_{nv}$	A_e (in ²)	8.59
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	251.37
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	418.95
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.33 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	163.92
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1495.81
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	11.88
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	56.41
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	356.25
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{tn} (kips)	534.38
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2538.28
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.73 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.2
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	163.9
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	7.4
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	9.125
Size of weld in 1/16 th s of an inch	D_{w3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1495.8
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.3
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{Exx})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.93 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.7
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	510.2

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.25
Top plate dimension (see schedule)	$P6$ (in)	30.75
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	38.438
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1153.1
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1153.1
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.44 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	9
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	21.563
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	840.9
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	630.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.81 OK

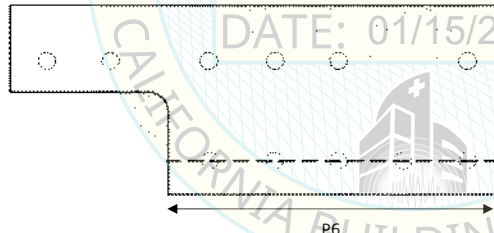


Fig. C-15.8. Shear rupture of net section on top plate.

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Total number of bolts (see step 5)	n_b	9
Number of bolts in the M region (see step 5)	n_m	4
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	226.8

Yielding check

Top plate dimension	$P4$ (in)	2
Top plate dimension	$P5$ (in)	3.25
Top plate thickness (see step 2)	t_p (in)	1.25
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	6.563
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	328.1
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	295.3
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.77 OK

Rupture check

Top plate dimension	$P4$ (in)	2
Top plate dimension	$P5$ (in)	3.25
Top plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	4.688
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	304.7
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	228.5
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99 OK

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.6
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3
Provided P2 dimension	$P2$ (in)	2.75 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Top plate thickness (see step 2)	t_p (in)	1.25
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.575
Inside length of the top plate	$P6$ (in)	30.75
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	15.117
Dimension, $([P6] - m) / 2$	e (in)	7.817
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-9.2
Provided P2 dimension	$P2$ (in)	2.75 OK

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.75 OK
Top plate thickness (see step 2)	t_p (in)	1.25
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.2 OK

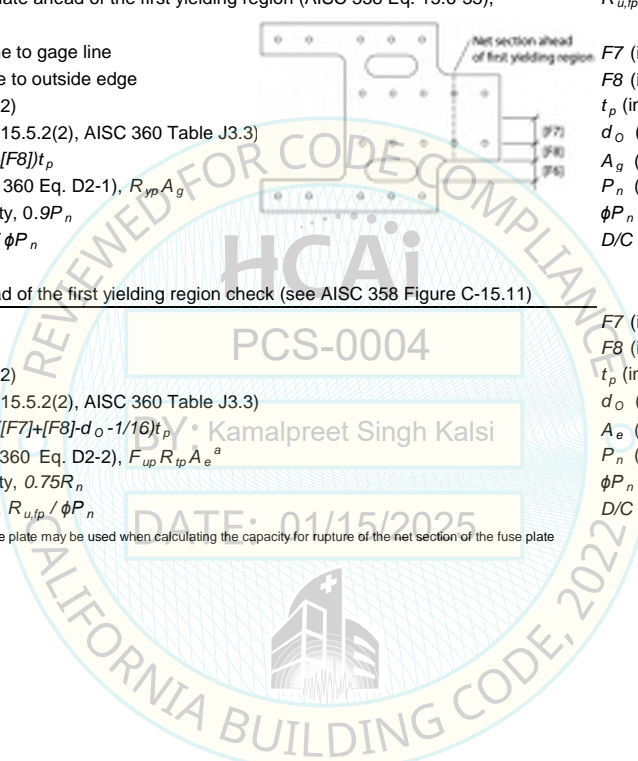
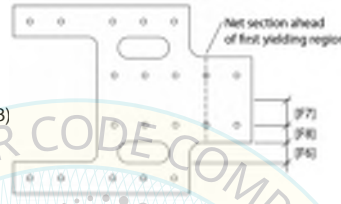
Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	7
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	3
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	345.2
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	3.075
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	14.563
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	728.1
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	655.3
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.53 OK

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	3.075
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	10.813
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e^a$	P_n (kips)	843.4
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	632.5
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.55 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Step 19. Determine the yielding region depth [F2]

Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	402.7
Fuse plate thickness(see step 2)	t_p (in)	1.25
Width of fuse plate cut-out	$F6$ (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{ip})B[F6])/(2t_p(0.6F_{up}R_{ip})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.806
Provided yielding region depth	$F2$ (in)	2.750 OK
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	206.3
Ratio	V_{fe} / V_y	2.0
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	1.00 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	7
Number of bolts in the M region	$n_{m, Bot}$	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	345.2
Yielding check		
Fuse plate dimension	$F4$ (in)	2
Fuse plate dimension	$F5$ (in)	1.875
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	9.6875
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	484.38
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	435.94
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	2
Fuse plate dimension	$F5$ (in)	1.875
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	5.9375
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	463.1
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	347.3
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.99 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_d/2$	P_d (kips)	107.5
External continuity plate dimension	C5 (in)	3.25
Column depth	d_c (in)	22.5
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	58.2
Cover plate overhang	C3 (in)	6
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕP_n (kips)	140.6
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	0.41 OK

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	58.2
Weld 5 size	D_{W5} (1/16 in)	5
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16([C3]-1)(2\text{ sides})$	ϕR_n (kips)	104.4
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.56 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	22.5
Beam depth	d_b (in)	24.7
Shear force from gravity	$V_{gravity}$ (kips)	40.1
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	163.9
Distance from alignment line to column	C1 (in)	9.125
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	60.6
Weld 7 size	D_{W7} (1/16 in)	4
Cover plate overhang	C3 (in)	6
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_w/16[C3](2\text{ sides})$	ϕR_n (kips)	66.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.91 OK

Bridge plate to column flange weld failure check

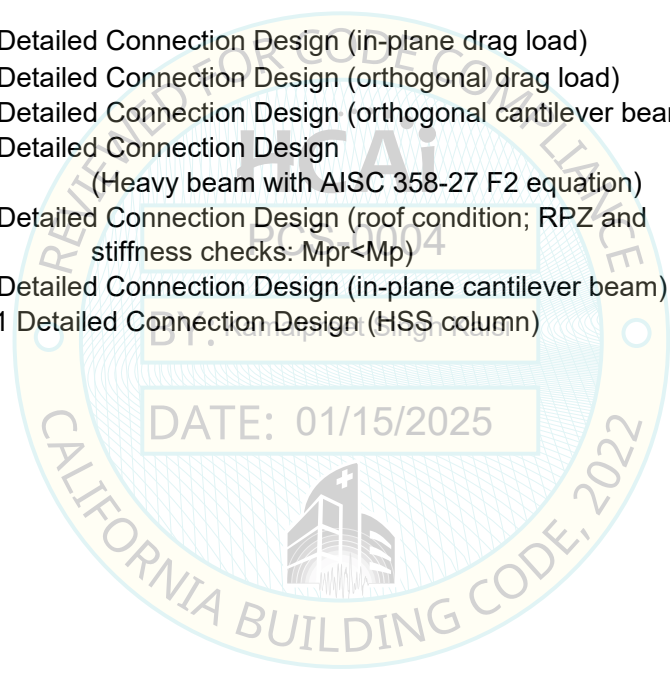
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	163.9
Distance from alignment line to column	C1 (in)	9.125
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	60.6
Weld 6 size	D_{W6} (1/16 in)	4
Cover plate overhang	C3 (in)	6
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	10.4
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	86.9
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.70 OK

Shear plate shear failure check (when $W1 > 3/4"$ without shear plates, shear plates are used)

Column depth	d_c (in)	22.5
Column flange thickness	t_{fc} (in)	1.36
Weld 4 size	D_{W4} (1/16 in)	9
Weld 4 effective length, $d_c - 2t_{fc}$	l_{eff} (in)	19.78
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16(l_{eff})$	R_u (kips)	247.8
Shear plate thickness, $t_{sp} \geq 1.1W4$ or $3/8"$	t_{sp} (in)	0.625
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c - 2t_{fc})$	ϕR_n (kips)	370.9
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.7 OK

4 ADDITIONAL EXAMPLE CONNECTIONS

DF150 Detailed Connection Design (in-plane drag load)	4.1
DF151 Detailed Connection Design (orthogonal drag load)	4.2
DF152 Detailed Connection Design (orthogonal cantilever beam)	4.3
DF153 Detailed Connection Design (Heavy beam with AISC 358-27 F2 equation)	4.4
DF250 Detailed Connection Design (roof condition; RPZ and stiffness checks: $M_{pr} < M_p$)	4.5
DF251 Detailed Connection Design (in-plane cantilever beam)	4.6
DHF101 Detailed Connection Design (HSS column)	4.7



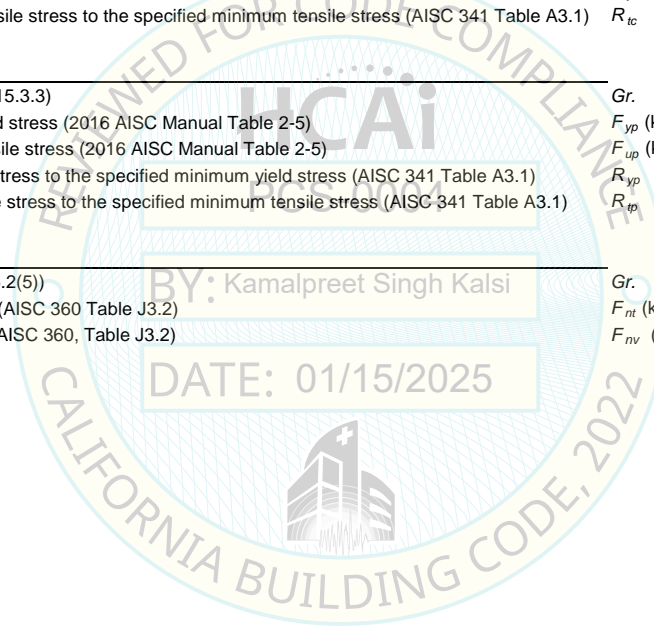
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF150
W21X122 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2
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Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84



BY: Kamalpreet Singh Kalsi

DATE: 01/15/2025

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W24X146
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	24.7 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	146 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	33.2 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_y)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	5.92 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))	See bracing calculations	

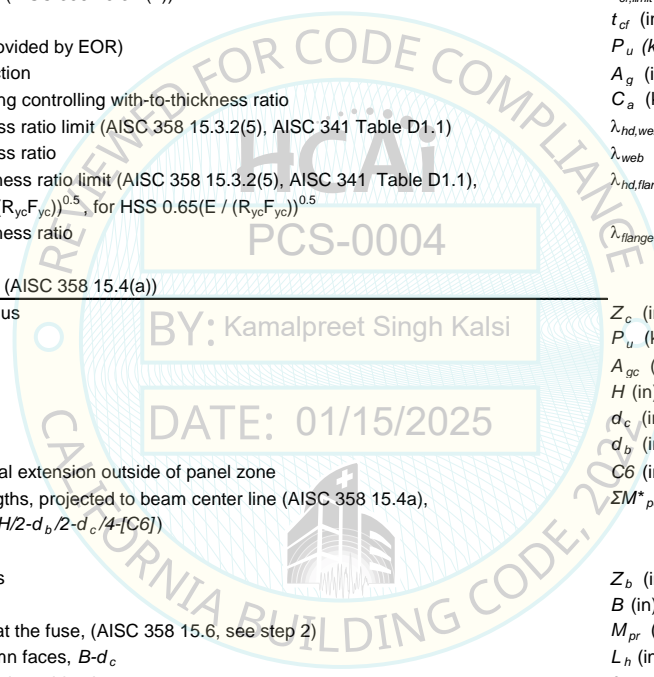
Column limitation checks

		W21X122
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	21.7
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	122 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.96 OK
Axial force on the column (provided by EOR)	P_u (kips)	271.9
Gross area of the column section	A_g (in ²)	35.9
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	51.1
Column web width-to-thickness ratio	λ_{web}	31.3 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	6.45 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	307
Column design axial load	P_u (kips)	271.9
Column area	A_{gc} (in ²)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	3.23E+04
Beam plastic section modulus	Z_b (in ³)	418
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900
Clear distance between column faces, $B - d_c$	L_h (in)	338.3
Live load factor in seismic load combination	f_l	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f_lL + 0.2S$	V_g (kips)	38.2
Beam shear (AISC 358 15.4a), $2M_{pr} / L_h + V_g$	V_b (kips)	161.7
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1755
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	2.27E+04
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	Ratio > 1.0	1.42 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1))



Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	8404
Beam plastic section modulus	Z_b (in ³)	418
Beam plastic bending moment, $F_y Z_b$	M_p (k-in)	20900
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	20900 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	404.6
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	404.6

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Beam area	A_{gb} (in ²)	43.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	107.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	512.1
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	56
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	573.5
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhang dimension	C3 (in)	3.25
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	28.2
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	634.5
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	634.5
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.81 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag condition	h_{cp} (in)	30.70
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	690.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	690.8
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.83 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.52
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.60 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Beam plastic section modulus	Z_b (in ³)	418
Beam flange thickness	t_{fb} (in)	1.09
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	1.499
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	70.8
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	347.2
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	107.5
Bolt diameter	d_{bolt} (in)	1.125
Area of single bolt, $\pi*d_{bolt}^2/4$	A_{bolt} (in ²)	0.994
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b, req}$	6.1
Number of bolts per line (AISC 358 15.2(1))	n_b	9 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.375
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.125
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.843
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p, min}$	3.2
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	5 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	512.1
Beam flange thickness	t_{fb} (in)	1.09
Fuse and top plate thickness	$T2$ (in)	1.125
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	1.1
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	1721.7
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	1317.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o/2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	77.0
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1394.9
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	751.5
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	751.5
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	563.6
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91 OK

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068
Bolt group demand from wind per line, $M_u/(2(d_b + t_p))$	R_u (kip)	59.4
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	9
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	244.1
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	207.5
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.29 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

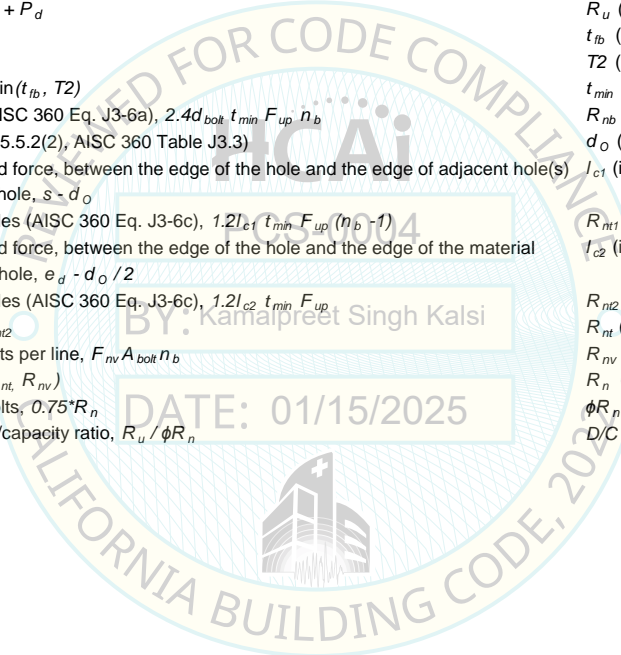
- Maximum probable force per bolt line (see step 2)
- Total drag force through fuse plate (all drag force transmitted through top plate)
- Drag force per line of bolts, $P_{d,total}/2$
- Bolt diameter
- Area of single bolt, $\pi \cdot d_{bolt}^2 / 4$
- Bolt shear strength, $\phi F_{nv} A_b$
- Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
- Number of bolts per line on bottom flange (AISC 358 15.2(1))
- Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
- Bolt spacing (see schedule)
- Plate thickness, t_p (see T2 in drawings)
- Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
- Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2[F2_a] + 3in - 2e_d] / s + 1$
- Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
- Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
(AISC 358 Eq. 15.6-8), $nb - np$

V_{fe} (kips)	404.6
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	1.125
A_{bolt} (in ²)	0.994
ϕr_n (kips)	83.5
$n_{b, req}$	4.8
$n_{b, Bot}$	7 OK
e_d (in)	1.625
s (in)	3.375
t_p (in)	1.125
$F2_a$ (in)	3.843
$n_{p,min}$	3.2
$n_{p, Bot}$	4 OK
$n_{m, Bot}$	3

Flange bolt limit states check

- Total force on bolts per line, $V_{fe} + P_d$
- Beam flange thickness
- Fuse and top plate thickness
- Governing material thickness, $\min(t_{fb}, T2)$
- Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
- Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
- Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 I_{c1} is based on an oversized hole, $s - d_o$
- Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c1} t_{min} F_{up} (n_b - 1)$
- Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 I_{c2} is based on an oversized hole, $e_d - d_o / 2$
- Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c2} t_{min} F_{up}$
- Total tearout strength, $R_{nt1} + R_{nt2}$
- Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
- Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
- Factored shear capacity of all bolts, $0.75 \cdot R_n$
- Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	404.6
t_{fb} (in)	1.09
$T2$ (in)	1.125
t_{min} (in)	1.1
R_{nb} (kips)	1721.7
d_o (in)	1.4375
I_{c1} (in)	1.9
R_{nt1} (kips)	988.4
I_{c2} (in)	0.9
R_{nt2} (kips)	77.0
R_{nt} (kips)	1065.4
R_{nv} (kips)	584.5
R_n (kips)	584.5
ϕR_n (kips)	438.4
D/C	0.92 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.7
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	1
Cover plate overhang dimension	C3 (in)	3.25
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.357
Distance from alignment line to column	C1 (in)	6.375 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	107.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	56
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.125
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.7
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	3.375
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.70
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.88
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	6
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	1.28
Vertical distance from center of rotation to geometric center	e_y (in)	2.07

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) * L$ ΣR_L (k-in) 18212

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R * L_R$ (k-in)
W1RT	19.18	9.17	60.07	176.0
W4TL	20.38	5.27	32.57	107.5
W1LB	17.40	8.63	45.82	150.2
W4BR	15.51	5.23	32.86	81.1
W1RC	9.86	6.13	13.90	60.5
W4TC	17.23	4.15	4.91	71.5
W1LC	12.36	6.25	11.05	77.2
W4BC	13.11	4.00	6.46	52.5
		9.17	60.07	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	9.2
Orientation of Resultant Force	θ_r (degrees)	60.1
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma R_L)$	r_{uv} (kip/in)	10.0
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c * l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	10.0
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.27
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 * 0.6 * 70 * (1 + 0.5 * \sin^{1.5}(\theta_r)) * (p_i * (1.9 - 0.9 * p_i)))^{0.3} * 0.707 * D_{W1} / 16$	ϕr_n (kips)	11.58
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.87 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	C3 (in)	3.25
External continuity plate dimension	C5 (in)	3.375
Weld 2 size	D_{W2} (1/16 in)	10
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.63
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.34
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	18.2
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3]) - 2l_{hb2} - l_{we2,eff})$	r_{un2} (kip/in)	17.6
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	25.3
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2 \text{ sides})$	ϕR_n (kip/in)	27.8
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.91 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1728.5
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	512.1
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c + 2[C3])t_p$	ϕR_n (kips)	951.8
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	10064.8
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.71 OK

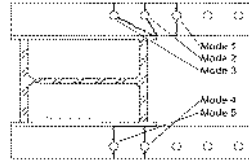
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column fillet dimension	k (in)	1.5
Column fillet dimension	$k1$ (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	91.5
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	10
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1697.4
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	809.2
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	2656.5
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1992.4
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.26 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	5.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	284.5
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.625
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Net area of external continuity plate cross section, $t_p([C4]-d_{hole,std}+1/16)$	A_n (in ²)	5.977
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	388.5
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	291.4
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

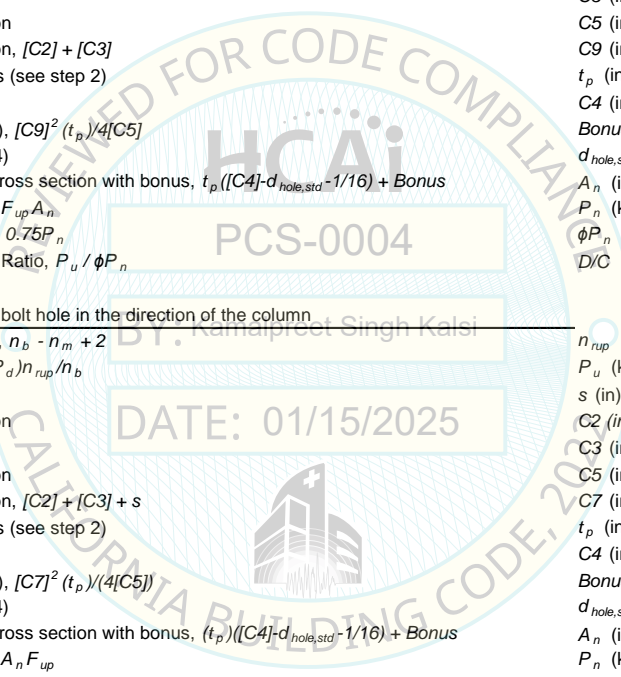
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	341.4
External continuity plate dimension	$C2$ (in)	2.625
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate dimension	$C5$ (in)	3.375
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	5.875
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.625
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p)/4[C5]$	$Bonus$ (in)	2.876
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	8.853
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	575.4
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	431.6
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.79 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	398.3
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	2.625
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate dimension	$C5$ (in)	3.375
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	9.25
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.625
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p)/(4[C5])$	$Bonus$ (in)	7.130
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $(t_p)/([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	13.107
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	851.9
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	639.0
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.62 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	341.4
External continuity plate dimension	$C2$ (in)	2.625
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate width	$C4$ (in)	6.625
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	5.875
External continuity plate thickness (see step 2)	t_p (in)	1.125
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	7.453
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	6.609
Net tension area, $A_{gt} - ((t_p)^2(d_{hole,std}+1/16))$	A_{nt} (in ²)	5.977
Net shear area, A_{gv}	A_{nv} (in ²)	6.609
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	586.8
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	440.1
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.78 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 5 rupture demand, $(V_{fe} + P_d) \cdot n_{rup} / n_b$	R_u (kips)	398.3
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	C2 (in)	2.625
Cover plate overhang dimension	C3 (in)	3.25
External continuity plate width	C4 (in)	6.625
External continuity plate dimension, $[C2] + [C3] + s$	C7 (in)	9.25
External continuity plate thickness (see step 2)	t_p (in)	1.125
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.25
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	7.453
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	10.406
Net tension area, $A_{gt} - (t_p (d_{hole} + 1/16))$	A_{nt} (in ²)	5.977
Net shear area, A_{gv}	A_{nv} (in ²)	10.406
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	700.7
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	525.5
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.76 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	38.2
Maximum probable moment at fuse location (see Step 2)	M_{pr} (K-in)	20900
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	161.7
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	l_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_wC_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.34 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	107.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1024.3
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	9
Length of connection on beam	B4 (in)	28.625
Distance from beam centerline to bolt line	P2 (in)	3
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	7.521
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	62.403
Net tension area, $A_{gt} - (t_{fb} (d_{hole, std} + 1/16))$	A_{nt} (in ²)	6.090
Net shear area, $A_{gv} - (t_{fb} (d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	38.082
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1881.1
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1410.8
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.73 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

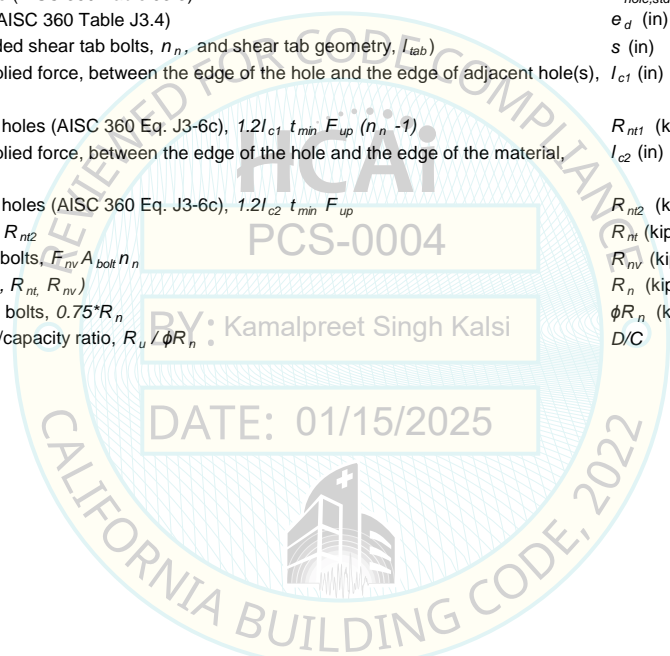
The required strength of shear tab bolt group (see step 10)	R_u (kips)	161.7
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.994
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	$T3$ (in)	0.5
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	351.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	477.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	34.1
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	511.9
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.65 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7
Distance from alignment line to column	$C1$ (in)	6.375
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	1031.1

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16)t_{tab}$	A_{nv} (in ²)	6.88
Effective net area, $A_e = A_{nv}$	A_e (in ²)	6.88
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up}A_{nv}$	ϕR_{nv} (kips)	201.09
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up}A_e$	ϕR_{tn} (kips)	335.16
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.65 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.75
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	1031.15
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp}A_{gv}$	ϕR_{ny} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp}A_{gv}$	ϕR_{nt} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nt})^2 + (V_u/\phi R_{ny})^4$	D/C	0.76 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.45
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	7.2
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	6.375
Size of weld in 1/16 ^{ths} of an inch	D_{w3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	1031.1
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.2
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.2
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.92 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.7
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	512.1

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.125
Top plate dimension (see schedule)	$P6$ (in)	31
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	34.875
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1046.3
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1046.3
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.49 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	9
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	19.688
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	767.8
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	575.9
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89 OK



Fig. C-15.8. Shear rupture of net section on top plate.

Note: For the alternate top plate, this check is conservative and does not need to be checked for the top plate.

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	512.1

Yielding check (LS 1)

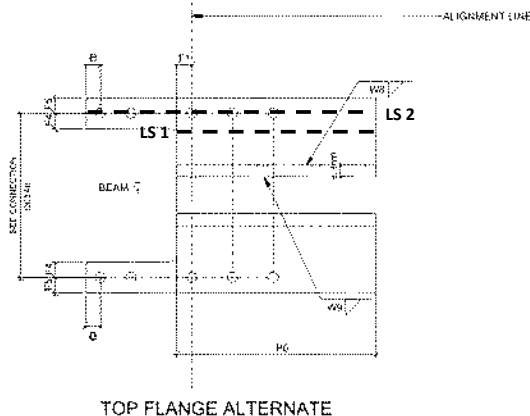
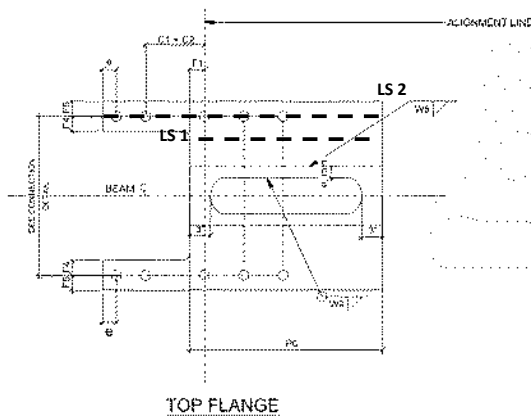
Top plate thickness (see step 2)	t_p (in)	1.125
Top plate dimension (see schedule)	$P6$ (in)	31
Fuse plate dimension	$F1$ (in)	2.00
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	34.875
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1046.3
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1046.3
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.49 OK

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Number of oversized bolts in Zone M (see step 5)	n_m	4
Number of oversized bolts on top plate (see step 5)	n_b	9
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Bolt spacing (see schedule)	s (in)	3.375
Fuse plate dimension	$F1$ (in)	2.00
Top plate dimension, $e_d + s*(n_m-1) + C1 + C2 + -F1 + P6$	$Dim 1$ (in)	49.625
Gross shear area of top plate, $(Dim 2)(t_p)$	A_g (in ²)	55.828
Number of oversized bolts on top plate (see step 5)	n_b	9
Net shear area of top plate, $A_g - (t_p d_o n_b)$	A_n (in ²)	41.273
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	1609.7
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	1207.2
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.42 OK

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u (kips)	512.1
Beam flange width	b_{fb} (in)	12.9
Weld 8 size	D_{W8} (1/16 in)	4
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.25
Weld 8 length, $P6 - 2l_{hb8}$	l_{w8} (in)	30.50
Weld 9 size	D_{W9} (1/16 in)	4
Weld 9 length, $l_{w8} - 2*3"$	l_{w9} (in)	24.50
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u / (l_{w8} + l_{w9})$	r_{uv} (kip/in)	9.3
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16 + D_{W9}/16)(1")$	ϕr_n (kip/in)	11.1
Alternative top plate-to-beam flange weld tearing check, $r_{uv} / \phi r_n$	D/C	0.84 OK



Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Total number of bolts (see step 5)	n_b	9
Number of bolts in the M region (see step 5)	n_m	4
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	227.6

Yielding check

Top plate dimension	$P4$ (in)	2.125
Top plate dimension	$P5$ (in)	3.625
Top plate thickness (see step 2)	t_p (in)	1.125
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	6.469
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	323.4
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	291.1
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.78 OK

Rupture check

Top plate dimension	$P4$ (in)	2.125
Top plate dimension	$P5$ (in)	3.625
Top plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	4.781
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_u A_e$	P_n (kips)	310.8
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	233.1
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 OK

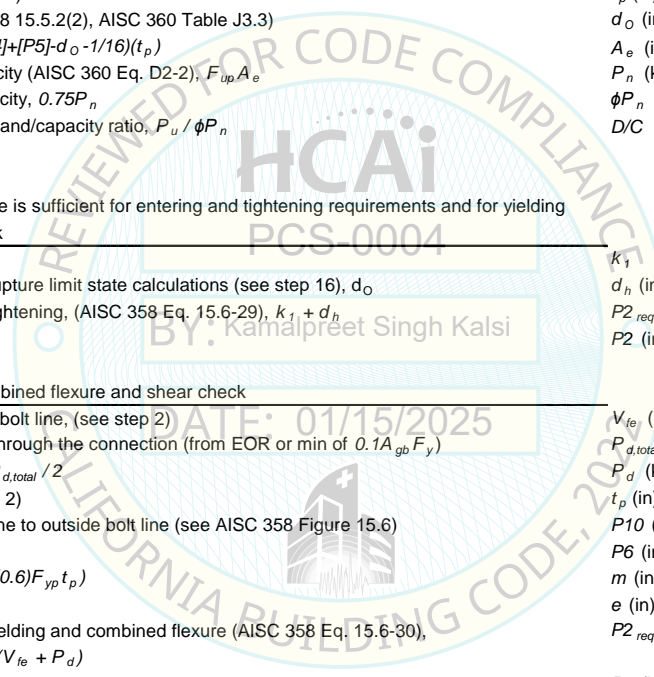
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.6
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3
Provided P2 dimension	$P2$ (in)	3 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Top plate thickness (see step 2)	t_p (in)	1.125
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.325
Inside length of the top plate	$P6$ (in)	31.00
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	16.861
Dimension, $([P6] - m) / 2$	e (in)	7.070
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-6.4
Provided P2 dimension	$P2$ (in)	3 OK



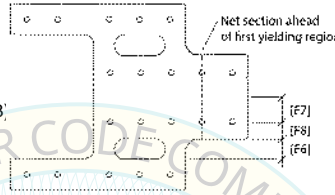
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.75 OK
Top plate thickness (see step 2)	t_p (in)	1.125
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.4 OK

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

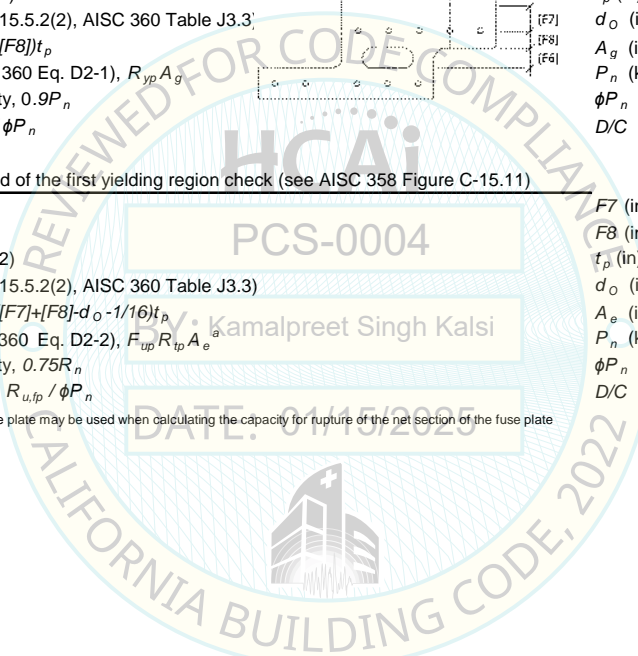
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	7
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	3
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	346.8
Fuse plate dimension, centerline to gage line	$F7$ (in)	3
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.45
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	12.263
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	613.1
Factored tensile capacity, $0.9P_n$	ϕP_n (kips)	551.8
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.63 OK



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	3
Fuse plate dimension	$F8$ (in)	2.45
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	8.888
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$	P_n (kips)	693.2
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	519.9
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.67 OK

*The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	404.6
Fuse plate thickness(see step 2)	t_p (in)	1.125
Width of fuse plate cut-out	$F6$ (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B/[F6])/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	3.147
Provided yielding region depth	$F2$ (in)	3.125 OK
Fuse yield force (one side), $2(0.6F_{yp})/[F2]t_p$	V_y (kips)	210.9
Ratio	V_{fe} / V_y	1.9
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	0.88 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	7
Number of bolts in the M region	$n_{m, Bot}$	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	346.8
Yielding check		
Fuse plate dimension	$F4$ (in)	2.125
Fuse plate dimension	$F5$ (in)	2.125
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	9.5625
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	478.13
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	430.31
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	2.125
Fuse plate dimension	$F5$ (in)	2.125
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	6.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	482.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	362.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.96 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	107.5
External continuity plate dimension	C5 (in)	3.375
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	69.3
Cover plate overhang	C3 (in)	3.25
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	1.000
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕR_n (kips)	101.3
Tension failure demand/capacity ratio, $P_u/\phi R_n$	D/C	0.68 OK

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	69.3
Weld 5 size	D_{W5} (1/16 in)	8
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W5}/16([C3]-1)(2 \text{ sides})$	ϕR_n (kips)	75.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.92 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Shear force from gravity	$V_{gravity}$ (kips)	38.2
Maximum probable moment at fuse location (see Step 2)	M_{pr} (K-in)	20900
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	161.7
Distance from alignment line to column	C1 (in)	6.375
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	41.7
Weld 7 size	D_{W7} (1/16 in)	5
Cover plate overhang	C3 (in)	3.25
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_{W7}/16[C3](2 \text{ sides})$	ϕR_n (kips)	45.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.92 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	161.7
Distance from alignment line to column	C1 (in)	6.375
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	41.7
Weld 6 size	D_{W6} (1/16 in)	5
Cover plate overhang	C3 (in)	3.25
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	6.5
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W6}/16(l_{eff})$	ϕR_n (kips)	67.9
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.62 OK

Shear plate shear failure check (when $W1 > 3/4"$ without shear plates, shear plates are used)

Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Weld 4 size	D_{W4} (1/16 in)	6
Weld 4 effective length, $d_c - 2 t_{fc}$	l_{eff} (in)	19.78
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16(l_{eff})$	R_u (kips)	165.2
Shear plate thickness, $t_{sp} \geq 1.1W4 \text{ or } 3/8"$	t_{sp} (in)	0.500
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c - 2t_{fc})$	ϕR_n (kips)	296.7
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6 OK

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

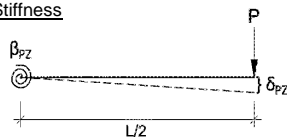
Show that effective panel zone stiffness $\geq 10 \times$ beam flexural stiffness

Effective Panel Zone Stiffness

$$K_{PZ} = P/\delta_{PZ}$$

$$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$$

$$K_{PZ} = \beta_{PZ}/(L/2)^2$$

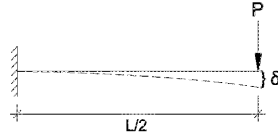


Beam Stiffness

$$K_B = P/\delta_B$$

$$\delta_B = P(L/2)^3/3EI$$

$$K_B = 3EI/(L/2)^3$$

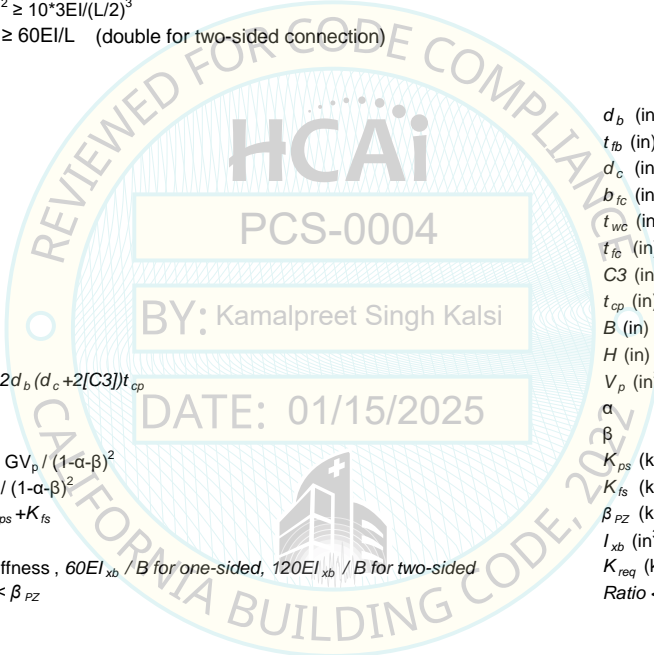


$$K_{PZ} \geq 10K_B$$

$$\beta_{PZ}/(L/2)^2 \geq 10 \times 3EI/(L/2)^3$$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column depth	d_c (in)	21.7
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.6
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang	C3 (in)	3.3
Cover plate thickness	t_{cp} (in)	0.75
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])t_{cp}$	V_p (in ³)	1366.4
Column ratio, $(d_c - t_{fc}) / B$	α	0.058
Beam ratio, $(d_b - t_{fb}) / H$	β	0.109
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	2.21E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.22E+07
Beam moment of inertia	I_{xb} (in ³)	4580
Required panel zone spring stiffness, $60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided	K_{req} (k-in)	2.21E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	Ratio < 1	1.00 OK



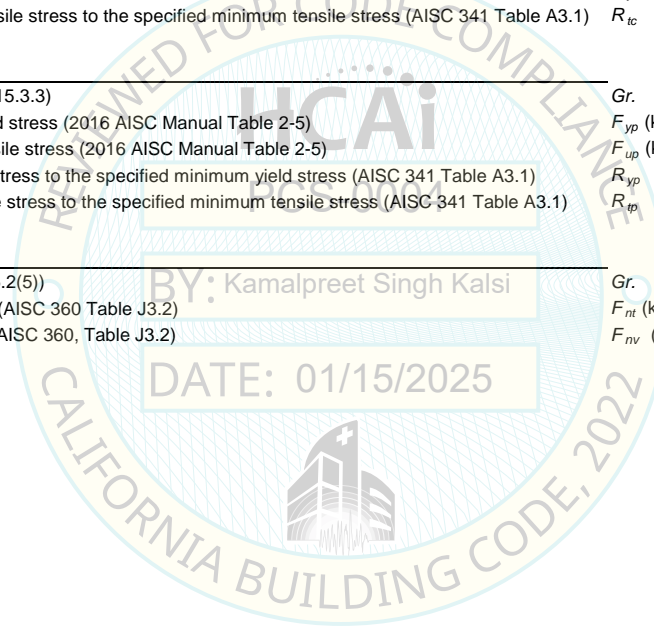
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF151
W21X122 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200
<hr/>		
Beams		
Beam material grade	$Gr.$	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1
<hr/>		
Columns		
Column material grade	$Gr.$	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	$Gr.$	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	$Gr.$	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W24X146
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	24.7 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	146 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	33.2 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_y)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	5.92 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))	See bracing calculations	

Column limitation checks

		W21X122
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	21.7
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	122 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.96 OK
Axial force on the column (provided by EOR)	P_u (kips)	271.9
Gross area of the column section	A_g (in ²)	35.9
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	51.1
Column web width-to-thickness ratio	λ_{web}	31.3 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	6.45 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	307
Column design axial load	P_u (kips)	271.9
Column area	A_{gc} (in ²)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	3.23E+04
Beam plastic section modulus	Z_b (in ³)	418
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900
Clear distance between column faces, $B - d_c$	L_h (in)	338.3
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	38.2
Beam shear (AISC 358 15.4a), $2M_{pr} / L_h + V_g$	V_b (kips)	161.7
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1755
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	2.27E+04
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	Ratio > 1.0	1.42 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1))

Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	8404
Beam plastic section modulus	Z_b (in ³)	418
Beam plastic bending moment, $F_y Z_b$	M_p (k-in)	20900
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	20900 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25
Beam depth	d_b (in)	24.7
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	402.7
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	402.7

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	402.7
Beam area	A_{gb} (in ²)	43.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	107.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	510.2
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	56
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25
Beam depth	d_b (in)	24.7
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	573.8
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.875
Cover plate overhang dimension	C3 (in)	3
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	27.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	727.1
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	727.1
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.70 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	26.45
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	694.3
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	694.3
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.83 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.52
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.86 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Beam plastic section modulus	Z_b (in ³)	418
Beam flange thickness	t_{fb} (in)	1.09
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	1.499
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	70.8
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	347.2
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	107.5
Bolt diameter	d_{bolt} (in)	1.125
Area of single bolt, $\pi*d_{bolt}^2/4$	A_{bolt} (in ²)	0.994
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b, req}$	6.1
Number of bolts per line (AISC 358 15.2(1))	n_b	9 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.375
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.25
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.442
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p, min}$	3.0
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	5 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	510.2
Beam flange thickness	t_{fb} (in)	1.09
Fuse and top plate thickness	$T2$ (in)	1.25
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	1.1
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	1721.7
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	1317.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o/2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	77.0
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1394.9
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	751.5
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	751.5
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	563.6
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91 OK

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068
Bolt group demand from wind per line, $M_u/(2(d_b + t_p))$	R_u (kip)	59.1
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	9
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	244.1
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	207.5
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.28 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

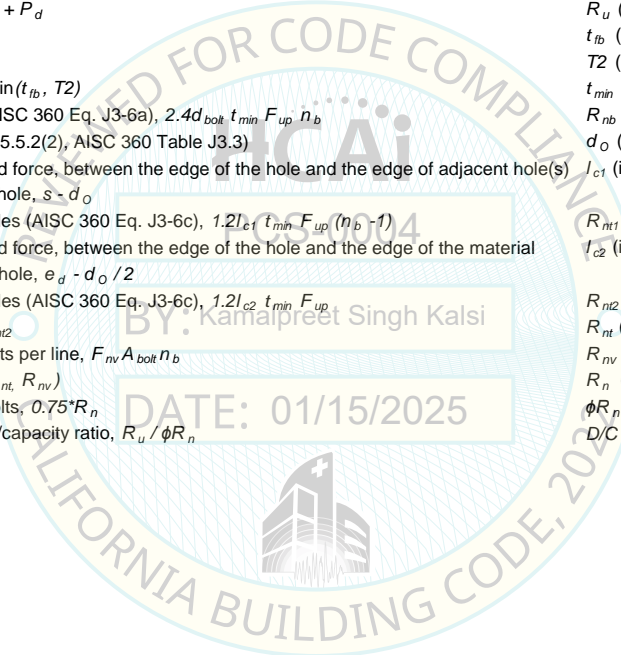
Maximum probable force per bolt line (see step 2)
 Total drag force through fuse plate (all drag force transmitted through top plate)
 Drag force per line of bolts, $P_{d,total}/2$
 Bolt diameter
 Area of single bolt, $\pi \cdot d_{bolt}^2 / 4$
 Bolt shear strength, $\phi F_{nv} A_b$
 Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
 Number of bolts per line on bottom flange (AISC 358 15.2(1))
 Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
 Bolt spacing (see schedule)
 Plate thickness, t_p (see T2 in drawings)
 Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
 Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2[F2_a] + 3in - 2e_d] / s + 1$
 Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
 Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
 (AISC 358 Eq. 15.6-8), $n_b - n_p$

V_{fe} (kips)	402.7
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	1.125
A_{bolt} (in ²)	0.994
ϕr_n (kips)	83.5
$n_{b, req}$	4.8
$n_{b, Bot}$	7 OK
e_d (in)	1.625
s (in)	3.375
t_p (in)	1.25
$F2_a$ (in)	3.442
$n_{p,min}$	3.0
$n_{p, Bot}$	4 OK
$n_{m, Bot}$	3

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$
 Beam flange thickness
 Fuse and top plate thickness
 Governing material thickness, $\min(t_{fb}, T2)$
 Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
 Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
 Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 I_{c1} is based on an oversized hole, $s - d_o$
 Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c1} t_{min} F_{up} (n_b - 1)$
 Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 I_{c2} is based on an oversized hole, $e_d - d_o / 2$
 Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c2} t_{min} F_{up}$
 Total tearout strength, $R_{nt1} + R_{nt2}$
 Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
 Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
 Factored shear capacity of all bolts, $0.75 \cdot R_n$
 Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	402.7
t_{fb} (in)	1.09
$T2$ (in)	1.25
t_{min} (in)	1.1
R_{nb} (kips)	1721.7
d_o (in)	1.4375
I_{c1} (in)	1.9
R_{nt1} (kips)	988.4
I_{c2} (in)	0.9
R_{nt2} (kips)	77.0
R_{nt} (kips)	1065.4
R_{nv} (kips)	584.5
R_n (kips)	584.5
ϕR_n (kips)	438.4
D/C	0.92 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.7
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	1
Cover plate overhang dimension	C3 (in)	3
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max}d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.107
Distance from alignment line to column	C1 (in)	6.125 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	107.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	56
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.25
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.875
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	26.45
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	3.125
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5625
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	25.33
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	5.13
Weld 1 size	D_{W1} (1/16 in)	7
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	7
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	1.21
Vertical distance from center of rotation to geometric center	e_y (in)	1.95

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) \cdot L$ ΣR_L (k-in) 18803

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R \cdot L_R$ (k-in)
W1RT	17.30	8.94	56.13	154.7
W4TL	20.24	6.08	32.59	123.0
W1LB	15.96	8.33	40.93	133.0
W4BR	15.65	6.16	32.83	96.4
W1RC	9.89	6.22	12.89	61.5
W4TC	17.11	4.95	4.72	84.6
W1LC	12.26	6.34	10.37	77.7
W4BC	13.22	4.79	6.11	63.4
		8.94	56.13	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	8.9
Orientation of Resultant Force	θ_r (degrees)	56.1
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.51
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma R_L)$	r_{uv} (kip/in)	11.1
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c \cdot l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	11.1
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.29
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 \cdot 0.6 \cdot 70 \cdot (1 + 0.5 \cdot \sin^{1.5}(\theta_r)) \cdot (p_i \cdot (1.9 - 0.9p_i))^{0.3} \cdot 0.707 \cdot D_{W1} / 16)$	ϕr_n (kips)	13.24
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.84 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	C3 (in)	3
External continuity plate dimension	C5 (in)	3.125
Weld 2 size	D_{W2} (1/16 in)	10
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.63
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.21
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	18.4
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3]) - 2l_{hb2} - l_{we2,eff})$	r_{un2} (kip/in)	17.0
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	25.1
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2 \text{ sides})$	ϕR_n (kip/in)	27.8
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.90 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	402.7
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1594.4
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	510.2
External continuity plate thickness (see step 2)	t_p (in)	1.250
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c + 2[C3])t_p$	ϕR_n (kips)	1038.8
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	10790.0
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.64 OK

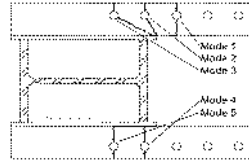
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column fillet dimension	k (in)	1.5
Column fillet dimension	$k1$ (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	90.5
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	10
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1678.8
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	809.2
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	2640.7
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1980.6
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.26 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	5.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	283.4
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	$C4$ (in)	6.625
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Net area of external continuity plate cross section, $t_p([C4]-d_{hole,std}+1/16)$	A_n (in ²)	6.641
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	431.6
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	323.7
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.88 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

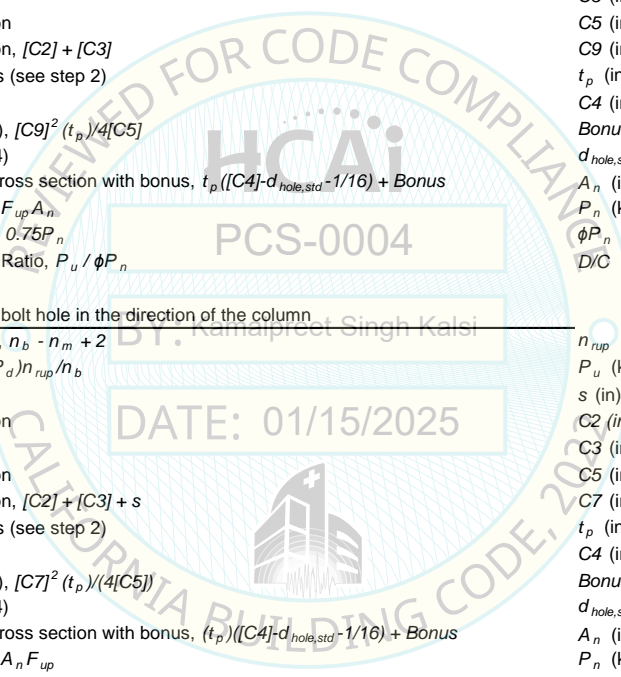
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	340.1
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate dimension	$C5$ (in)	3.125
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.125
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	$C4$ (in)	6.625
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p)/4[C5]$	$Bonus$ (in)	0.452
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	7.092
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	461.0
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	345.7
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.98 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	396.8
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate dimension	$C5$ (in)	3.125
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.5
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	$C4$ (in)	6.625
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p)/(4[C5])$	$Bonus$ (in)	3.025
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $(t_p)/([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	9.666
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	628.3
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	471.2
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.84 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	340.1
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate width	$C4$ (in)	6.625
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.125
External continuity plate thickness (see step 2)	t_p (in)	1.25
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	8.281
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	2.656
Net tension area, $A_{gt} - ((t_p)^2(d_{hole,std}+1/16))$	A_{nt} (in ²)	6.641
Net shear area, A_{gv}	A_{nv} (in ²)	2.656
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	511.3
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	383.5
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 5 rupture demand, $(V_{fe} + P_d) \cdot n_{rup} / n_b$	R_u (kips)	396.8
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	C2 (in)	-0.875
Cover plate overhang dimension	C3 (in)	3
External continuity plate width	C4 (in)	6.625
External continuity plate dimension, $[C2] + [C3] + s$	C7 (in)	5.5
External continuity plate thickness (see step 2)	t_p (in)	1.25
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.25
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	8.281
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	6.875
Net tension area, $A_{gt} - (t_p (d_{hole} + 1/16))$	A_{nt} (in ²)	6.641
Net shear area, A_{gv}	A_{nv} (in ²)	6.875
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	637.9
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	478.4
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.83 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	38.2
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	161.7
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	l_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_wC_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.34 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	215.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	107.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1020.4
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	9
Length of connection on beam	B4 (in)	28.625
Distance from beam centerline to bolt line	P2 (in)	3
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	7.521
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	62.403
Net tension area, $A_{gt} - (t_{fb} (d_{hole, std} + 1/16))$	A_{nt} (in ²)	6.090
Net shear area, $A_{gv} - (t_{fb} (d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	38.082
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1881.1
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1410.8
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.72 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

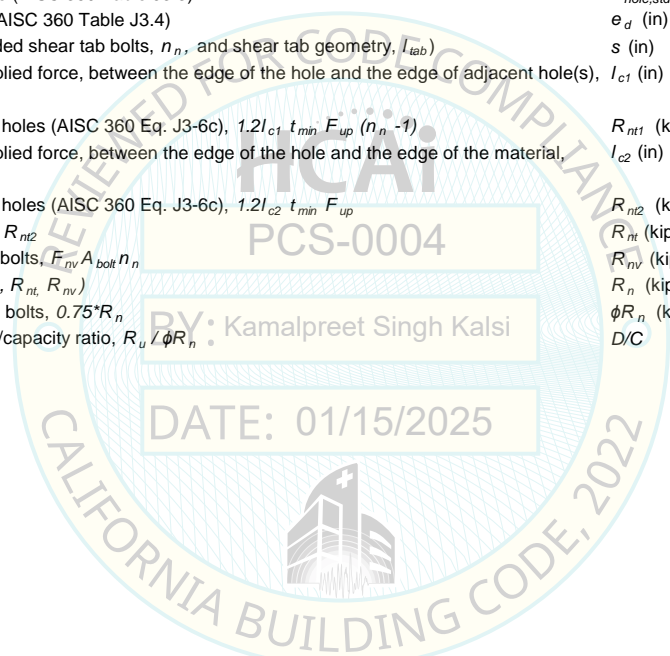
The required strength of shear tab bolt group (see step 10)	R_u (kips)	161.7
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.994
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	$T3$ (in)	0.5
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	351.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	477.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	34.1
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	511.9
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.65 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7
Distance from alignment line to column	$C1$ (in)	6.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	990.7

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	6.88
Effective net area, $A_e = A_{nv}$	A_e (in ²)	6.88
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	201.09
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	335.16
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.65 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.75
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	990.71
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{ny} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nt} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nt})^2 + (V_u/\phi R_{ny})^4$	D/C	0.74 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.2
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	7.3
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	6.125
Size of weld in 1/16 ^{ths} of an inch	D_{w3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	990.7
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u) / l_{w3}^2 - 2F_b n_{vb} / l_{w3}$ except when bridge plates present, $F_b n_{vb} / l_{w3}$	r_{un} (k/in)	7.3
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.3
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.93 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.7
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	510.2

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.25
Top plate dimension (see schedule)	$P6$ (in)	30.75
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	38.438
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1153.1
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1153.1
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.44 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	9
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	21.563
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	840.9
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	630.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.81 OK



Fig. C-15.8. Shear rupture of net section on top plate.

Note: For the alternate top plate, this check is conservative and does not need to be checked for the top plate.

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	510.2

Yielding check (LS 1)

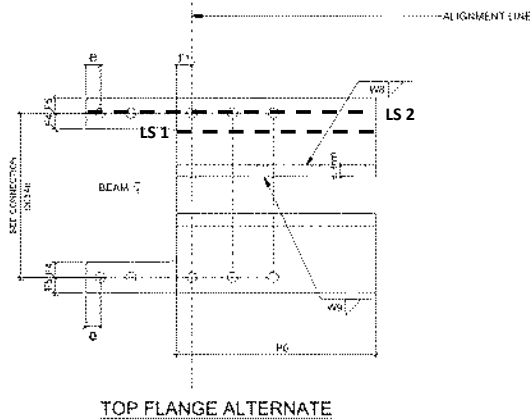
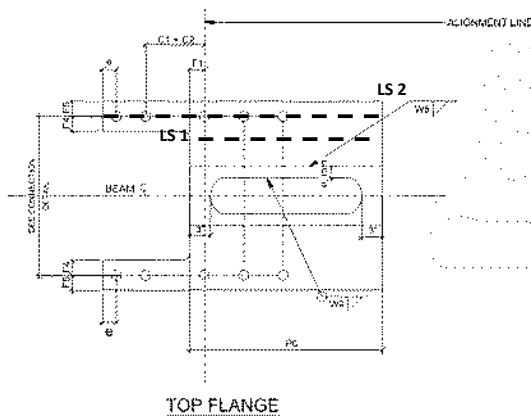
Top plate thickness (see step 2)	t_p (in)	1.25
Top plate dimension (see schedule)	$P6$ (in)	30.75
Fuse plate dimension	$F1$ (in)	1.88
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	38.438
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1153.1
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1153.1
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.44 OK

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Number of oversized bolts in Zone M (see step 5)	n_m	4
Number of oversized bolts on top plate (see step 5)	n_b	9
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Bolt spacing (see schedule)	s (in)	3.375
Fuse plate dimension	$F1$ (in)	1.88
Top plate dimension, $e_d + s*(n_m-1) + C1 + C2 + -F1 + P6$	$Dim 1$ (in)	45.75
Gross shear area of top plate, $(Dim 2)(t_p)$	A_g (in ²)	57.188
Number of oversized bolts on top plate (see step 5)	n_b	9
Net shear area of top plate, $A_g - (t_p d_o n_b)$	A_n (in ²)	41.016
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	1599.6
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	1199.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.43 OK

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u (kips)	510.2
Beam flange width	b_{fb} (in)	12.9
Weld 8 size	D_{W8} (1/16 in)	4
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.25
Weld 8 length, $P6 - 2l_{hb8}$	l_{w8} (in)	30.25
Weld 9 size	D_{W9} (1/16 in)	4
Weld 9 length, $l_{w8} - 2*3"$	l_{w9} (in)	24.25
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u / (l_{w8} + l_{w9})$	r_{uv} (kip/in)	9.4
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16 + D_{W9}/16)(1")$	ϕr_n (kip/in)	11.1
Alternative top plate-to-beam flange weld tearing check, $r_{uv} / \phi r_n$	D/C	0.84 OK



Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Total number of bolts (see step 5)	n_b	9
Number of bolts in the M region (see step 5)	n_m	4
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	226.8

Yielding check

Top plate dimension	$P4$ (in)	2
Top plate dimension	$P5$ (in)	3.25
Top plate thickness (see step 2)	t_p (in)	1.25
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	6.563
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	328.1
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	295.3
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.77 OK

Rupture check

Top plate dimension	$P4$ (in)	2
Top plate dimension	$P5$ (in)	3.25
Top plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	4.688
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_u A_e$	P_n (kips)	304.7
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	228.5
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99 OK

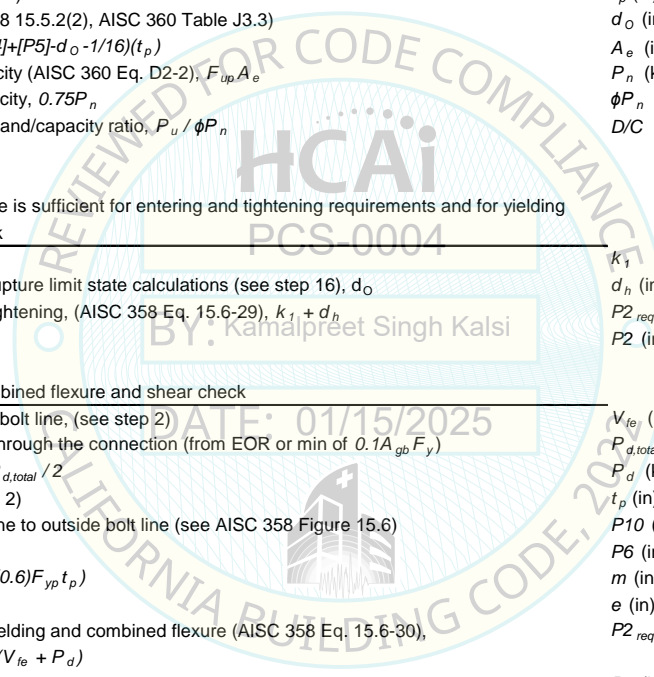
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.6
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3
Provided P2 dimension	$P2$ (in)	3 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Top plate thickness (see step 2)	t_p (in)	1.25
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.2
Inside length of the top plate	$P6$ (in)	30.75
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	15.117
Dimension, $([P6] - m) / 2$	e (in)	7.817
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-9.6
Provided P2 dimension	$P2$ (in)	3 OK



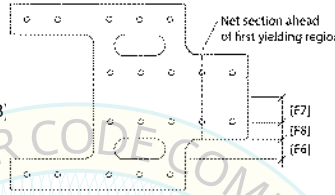
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.75 OK
Top plate thickness (see step 2)	t_p (in)	1.25
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.2 OK

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

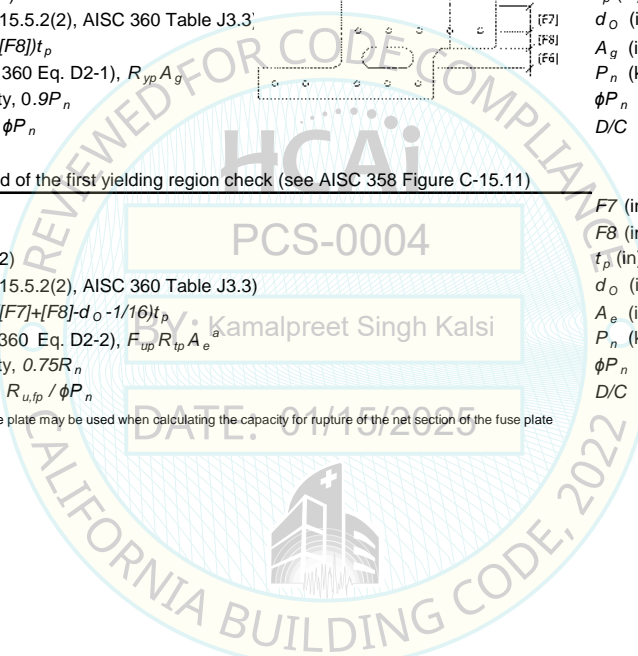
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	7
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	3
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	345.2
Fuse plate dimension, centerline to gage line	$F7$ (in)	3
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.45
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	13.625
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	681.3
Factored tensile capacity, $0.9P_n$	ϕP_n (kips)	613.1
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.56 OK



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	3
Fuse plate dimension	$F8$ (in)	2.45
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	9.875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$	P_n (kips)	770.3
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	577.7
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.60 OK

*The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	402.7
Fuse plate thickness(see step 2)	t_p (in)	1.25
Width of fuse plate cut-out	$F6$ (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6])/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.806
Provided yielding region depth	$F2$ (in)	2.750 OK
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	206.3
Ratio	V_{fe} / V_y	2.0
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	1.00 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	7
Number of bolts in the M region	$n_{m, Bot}$	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	345.2
Yielding check		
Fuse plate dimension	$F4$ (in)	2
Fuse plate dimension	$F5$ (in)	1.875
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	9.6875
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	484.38
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	435.94
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	2
Fuse plate dimension	$F5$ (in)	1.875
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	5.9375
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	463.1
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	347.3
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.99 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	107.5
External continuity plate dimension	C5 (in)	3.125
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	64.5
Cover plate overhang	C3 (in)	3
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	1.000
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕR_n (kips)	90.0
Tension failure demand/capacity ratio, $P_u/\phi R_n$	D/C	0.72 OK

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	64.5
Weld 5 size	D_{W5} (1/16 in)	8
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W5}/16([C3]-1)(2 \text{ sides})$	ϕR_n (kips)	66.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.97 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Shear force from gravity	$V_{gravity}$ (kips)	38.2
Maximum probable moment at fuse location (see Step 2)	M_{pr} (K-in)	20900
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	161.7
Distance from alignment line to column	C1 (in)	6.125
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	40.1
Weld 7 size	D_{W7} (1/16 in)	5
Cover plate overhang	C3 (in)	3
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_{W7}/16[C3](2 \text{ sides})$	ϕR_n (kips)	41.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.96 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	161.7
Distance from alignment line to column	C1 (in)	6.125
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	40.1
Weld 6 size	D_{W6} (1/16 in)	5
Cover plate overhang	C3 (in)	3
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	6
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W6}/16(l_{eff})$	ϕR_n (kips)	62.6
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.64 OK

Shear plate shear failure check (when $W1 > 3/4"$ without shear plates, shear plates are used)

Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Weld 4 size	D_{W4} (1/16 in)	7
Weld 4 effective length, $d_c - 2 t_{fc}$	l_{eff} (in)	19.78
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16(l_{eff})$	R_u (kips)	192.7
Shear plate thickness, $t_{sp} \geq 1.1W4 \text{ or } 3/8"$	t_{sp} (in)	0.500
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c - 2t_{fc})$	ϕR_n (kips)	296.7
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6 OK

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

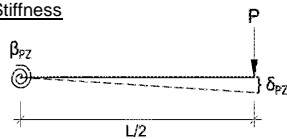
Show that effective panel zone stiffness $\geq 10 \times$ beam flexural stiffness

Effective Panel Zone Stiffness

$$K_{PZ} = P/\delta_{PZ}$$

$$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$$

$$K_{PZ} = \beta_{PZ}/(L/2)^2$$

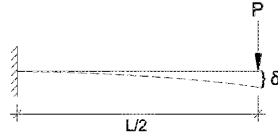


Beam Stiffness

$$K_B = P/\delta_B$$

$$\delta_B = P(L/2)^3/3EI$$

$$K_B = 3EI/(L/2)^3$$

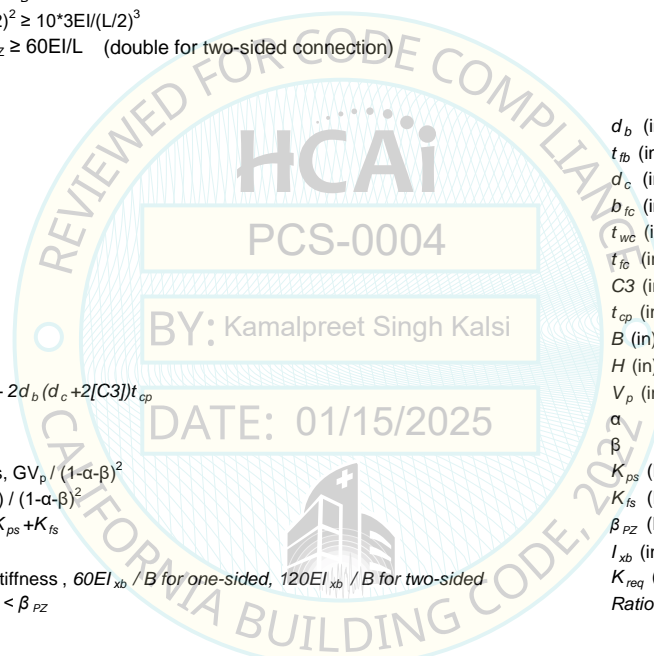


$$K_{PZ} \geq 10K_B$$

$$\beta_{PZ}/(L/2)^2 \geq 10 \times 3EI/(L/2)^3$$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column depth	d_c (in)	21.7
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.6
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang	C3 (in)	3.0
Cover plate thickness	t_{cp} (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])t_{cp}$	V_p (in ³)	1518.9
Column ratio, $(d_c - t_{fc})/B$	α	0.058
Beam ratio, $(d_b - t_{fb})/H$	β	0.109
Web and cover plate stiffness, $GV_p/(1-\alpha-\beta)^2$	K_{ps} (k-in)	2.45E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2)/(1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.47E+07
Beam moment of inertia	I_{xb} (in ³)	4580
Required panel zone spring stiffness, $60EI_{xb}/B$ for one-sided, $120EI_{xb}/B$ for two-sided	K_{req} (k-in)	2.21E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	Ratio < 1	0.90 OK



Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)

In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	107.5
Total orthogonal drag load (from EOR)	$P_{d,do,total}$ (kips)	50.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total} / 2$	$P_{d,do}$ (kips)	25.0
Moment from orthogonal connection (seismic load combination) (from EOR)	$M_{u,o}$ (k-in)	0
Orthogonal beam size		W24X55
Orthogonal beam depth	$d_{b,o}$ (in)	23.6
Orthogonal beam flange thickness	$t_{fb,o}$ (in)	0.51
Flange force associated with $M_{u,o}$, $M_{u,o} / (d_{b,o} - t_{fb,o})$	$P_{d,mo}$ (kips)	0.0
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	25.0
Column flange width	b_{fc} (in.)	12.4
Column depth	d_c (in.)	21.7
Column flange thickness	t_{fc} (in.)	0.96
Column web thickness	t_{wc} (in.)	0.60
Plate thickness (see T2 in drawings)	t_p (in)	1.25
External continuity plate gage dimension	C5 (in.)	3.13
Plate-to column flange force, $[(\Sigma V_{fe} + P_d) / (1+0.5t_{wc} / t_p)]$	R_{cp} (kips)	411.4
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	98.7
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	25.0

External continuity plate section check (at the coverplate)

Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	1594.4
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	25.0
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	510.2
Plate plastic section modulus, $(d_c - 2t_{fc})^2 t_p / 4$	Z (in ³)	122.27
Plate section area, $(d_c - 2t_{fc}) t_p$	A (in ²)	24.73
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	5501.9
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	1112.6
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	723.2
Interaction equation, $(M_r / M_c) + (P_r / P_c)^2 + (V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.54 OK
		Special detailing not required

External continuity plate section check (at the column web)

Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5]) - R_{cp}(b_{fc}/2)$	M_r (k-in)	2206.6
Normal demand at section, $=P_{d,o}$	P_r (kips)	25.0
Shear demand at section, $=R_{w1}$	V_r (kips)	98.7
Corner clip dimension	K_c (in)	1.46
Plate plastic section modulus, $(d_c - 2t_{fc} - 2K_c)^2 t_p / 4$	Z (in ³)	88.8
Plate section area, $(d_c - 2t_{fc} - 2K_c)t_p$	A (in ²)	21.08
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	3997.4
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	948.4
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	616.4
Interaction equation, $(M_r / M_c) + (P_r / P_c)^2 + (V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.55 OK
		Special detailing not required

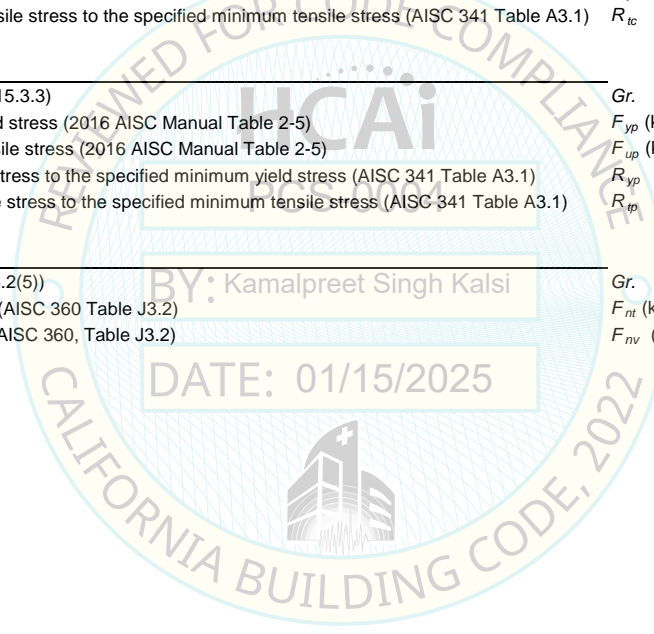
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF152
 W21X122 Column and W24X131 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



DATE: 01/15/2025

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W24X131
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	24.5 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	131 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	35.6 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_y)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	6.7 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))	<i>See bracing calculations</i>	

Column limitation checks

		W21X122
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	21.7
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	122 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.96 OK
Axial force on the column (provided by EOR)	P_u (kips)	53.5
Gross area of the column section	A_g (in ²)	35.9
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.0
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	57.2
Column web width-to-thickness ratio	λ_{web}	31.3 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	6.45 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	307
Column design axial load	P_u (kips)	53.5
Column area	A_{gc} (in ²)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.5
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	3.68E+04
Beam plastic section modulus	Z_b (in ³)	370
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	18500
Clear distance between column faces, $B - d_c$	L_h (in)	338.3
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	7.0
Beam shear (AISC 358 15.4a), $2M_{pr} / L_h + V_g$	V_b (kips)	116.4
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1263
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	1.98E+04
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	<i>Ratio > 1.0</i>	1.86 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1))

Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	3422
Beam plastic section modulus	Z_b (in ³)	370
Beam plastic bending moment, $F_y Z_b$	M_p (k-in)	18500
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	18500 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.5
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	361.0
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	361.0

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	361.0
Beam area	A_{gb} (in ²)	38.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	96.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	457.5
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	35
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.5
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	500.7
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.875
Cover plate overhang dimension	C3 (in)	3
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	27.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	727.1
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	727.1
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.63 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	22.25
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	584.1
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	584.1
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.86 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	24.5
Beam flange thickness	t_{fb} (in)	0.96
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.58
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.471
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.86 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Beam plastic section modulus	Z_b (in ³)	370
Beam flange thickness	t_{fb} (in)	0.96
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	1.320
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	62.1
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	307.9
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	20350
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	22012
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.92 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	193.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	96.5
Bolt diameter	d_{bolt} (in)	1.125
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.994
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$	$n_{b, req}$	5.5
Number of bolts per line (AISC 358 15.2(1))	n_b	8 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.375
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.125
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	3.428
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a] + 3in - 2e_d] / s + 1$	$n_{p, min}$	3.0
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	4 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	457.5
Beam flange thickness	t_{fb} (in)	0.96
Fuse and top plate thickness	$T2$ (in)	1.125
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	1.0
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	1347.8
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	1015.6
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	67.9
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1083.4
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	668.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	668.0
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	501.0
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91 OK

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451
Bolt group demand from wind per line, $M_u / (2(d_b + t_p))$	R_u (kip)	28.3
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	8
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	217.0
Total factored bolt slip resistance, $0.85 R_n$	ϕR_n (kips)	184.4
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.15 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

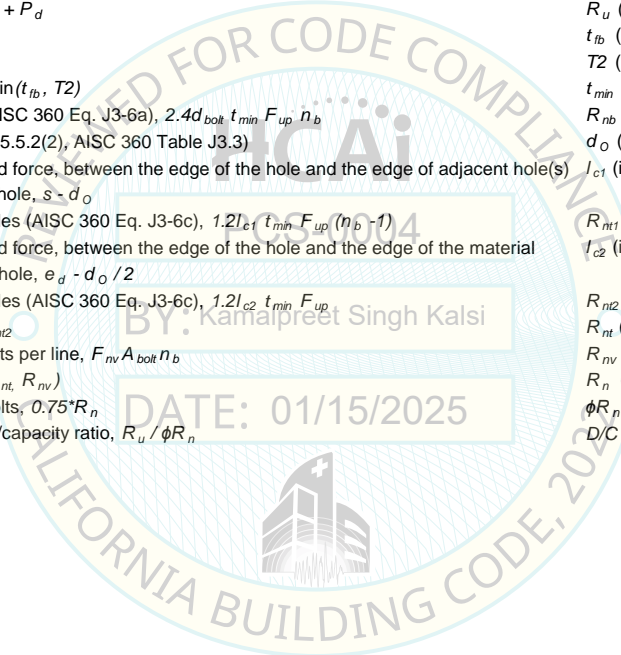
- Maximum probable force per bolt line (see step 2)
- Total drag force through fuse plate (all drag force transmitted through top plate)
- Drag force per line of bolts, $P_{d,total}/2$
- Bolt diameter
- Area of single bolt, $\pi \cdot d_{bolt}^2 / 4$
- Bolt shear strength, $\phi F_{nv} A_b$
- Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
- Number of bolts per line on bottom flange (AISC 358 15.2(1))
- Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
- Bolt spacing (see schedule)
- Plate thickness, t_p (see T2 in drawings)
- Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
- Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2[F2_a] + 3in - 2e_d] / s + 1$
- Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
- Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
(AISC 358 Eq. 15.6-8), $nb - np$

V_{fe} (kips)	361.0
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	1.125
A_{bolt} (in ²)	0.994
ϕr_n (kips)	83.5
$n_{b, req}$	4.3
$n_{b, Bot}$	6 OK
e_d (in)	1.625
s (in)	3.375
t_p (in)	1.125
$F2_a$ (in)	3.428
$n_{p,min}$	3.0
$n_{p, Bot}$	3 OK
$n_{m, Bot}$	3

Flange bolt limit states check

- Total force on bolts per line, $V_{fe} + P_d$
- Beam flange thickness
- Fuse and top plate thickness
- Governing material thickness, $\min(t_{fb}, T2)$
- Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
- Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
- Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 I_{c1} is based on an oversized hole, $s - d_o$
- Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c1} t_{min} F_{up} (n_b - 1)$
- Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 I_{c2} is based on an oversized hole, $e_d - d_o / 2$
- Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c2} t_{min} F_{up}$
- Total tearout strength, $R_{nt1} + R_{nt2}$
- Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
- Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
- Factored shear capacity of all bolts, $0.75 \cdot R_n$
- Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	361.0
t_{fb} (in)	0.96
$T2$ (in)	1.125
t_{min} (in)	1.0
R_{nb} (kips)	1347.8
d_o (in)	1.4375
I_{c1} (in)	1.9
R_{nt1} (kips)	725.4
I_{c2} (in)	0.9
R_{nt2} (kips)	67.9
R_{nt} (kips)	793.3
R_{nv} (kips)	501.0
R_n (kips)	501.0
ϕR_n (kips)	375.7
D/C	0.96 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.5
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	1
Cover plate overhang dimension	C3 (in)	3
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max}d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.095
Distance from alignment line to column	C1 (in)	6.125 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	96.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	35
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.5
Plate thickness (see T2 on drawings)	t_p (in)	1.125
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.875
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	22.25
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	3
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5625
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	21.13
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	5.00
Weld 1 size	D_{W1} (1/16 in)	7
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	7
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.84
Vertical distance from center of rotation to geometric center	e_y (in)	2.01

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) \cdot L$ ΣR_L (k-in) 16619

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R \cdot L_R$ (k-in)
W1RT	15.90	7.23	50.99	115.1
W4TL	20.00	5.88	31.77	117.5
W1LB	14.36	6.71	35.53	96.4
W4BR	15.72	6.21	34.27	97.7
W1RC	10.25	5.36	12.49	54.9
W4TC	17.04	4.98	3.49	84.8
W1LC	11.90	5.41	10.74	64.4
W4BC	13.04	4.82	4.56	62.8
		7.23	50.99	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	7.2
Orientation of Resultant Force	θ_r (degrees)	51.0
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.42
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma R_L)$	r_{uv} (kip/in)	10.8
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c \cdot l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	10.8
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.32
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 \cdot 0.6 \cdot 70 \cdot (1 + 0.5 \cdot \sin^{1.5}(\theta_r)) \cdot (p_i \cdot (1.9 - 0.9p_i)))^{0.3} \cdot 0.707 \cdot D_{W1}/16$	ϕr_n (kips)	12.84
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.84 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	C3 (in)	3
External continuity plate dimension	C5 (in)	3
Weld 2 size	D_{W2} (1/16 in)	8
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.50
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.34
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	16.5
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3]) - 2l_{hb2} - l_{we2,eff})$	r_{un2} (kip/in)	14.2
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	21.8
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2 \text{ sides})$	ϕR_n (kip/in)	22.3
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.98 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	361.0
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1372.4
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	457.5
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c + 2[C3])t_p$	ϕR_n (kips)	934.9
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	9711.0
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.63 OK

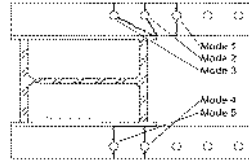
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5
Column fillet dimension	k (in)	1.5
Column fillet dimension	$k1$ (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	91.0
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	8
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1350.5
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	647.3
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	2118.9
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1589.2
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.29 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	4.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	228.7
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.25
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Net area of external continuity plate cross section, $t_p([C4]-d_{hole,std}+1/16)$	A_n (in ²)	5.555
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	361.1
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	270.8
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.84 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

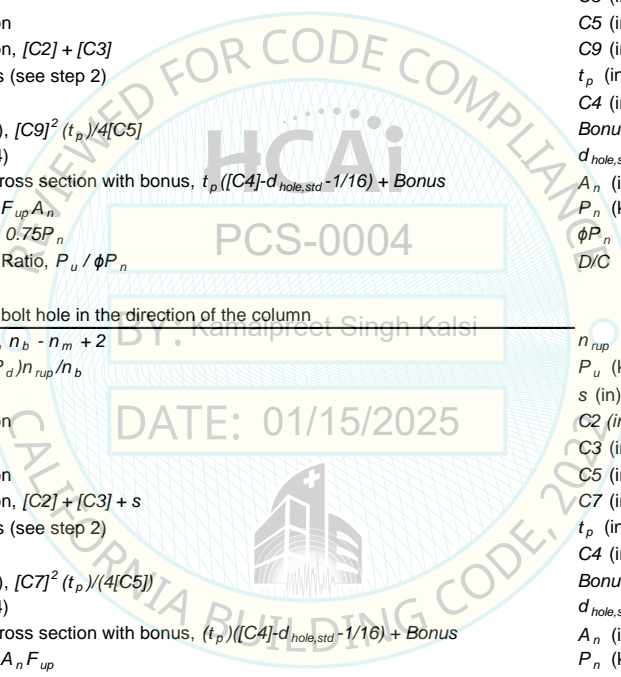
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	5
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	285.9
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate dimension	$C5$ (in)	3
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.125
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.25
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p)/4[C5]$	$Bonus$ (in)	0.423
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	5.978
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	388.6
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	291.4
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.98 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	6
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	343.1
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate dimension	$C5$ (in)	3
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.5
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.25
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p)/(4[C5])$	$Bonus$ (in)	2.836
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $(t_p)/([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	8.391
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	545.4
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	409.0
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.84 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	5
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	285.9
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate width	$C4$ (in)	6.25
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.125
External continuity plate thickness (see step 2)	t_p (in)	1.125
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	7.031
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	2.391
Net tension area, $A_{gt} - ((t_p)^2(d_{hole,std}+1/16))$	A_{nt} (in ²)	5.555
Net shear area, A_{gv}	A_{nv} (in ²)	2.391
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	432.8
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	324.6
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.88 OK



Step 9. (continued)

<u>Mode 5: Block shear through the second bolt hole in the direction of the column</u>		
Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	6
Mode 5 rupture demand, $(V_{fe} + P_d) \cdot n_{rup} / n_b$	R_u (kips)	343.1
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	C2 (in)	-0.875
Cover plate overhang dimension	C3 (in)	3
External continuity plate width	C4 (in)	6.25
External continuity plate dimension, $[C2] + [C3] + s$	C7 (in)	5.5
External continuity plate thickness (see step 2)	t_p (in)	1.125
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.25
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	7.031
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	6.188
Net tension area, $A_{gt} - (t_p (d_{hole} + 1/16))$	A_{nt} (in ²)	5.555
Net shear area, A_{gv}	A_{nv} (in ²)	6.188
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up} A_{nv} + F_{up} A_{nt} < 0.6F_{yp} A_{gv} + F_{up} A_{nt}$	R_n (kips)	546.7
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	410.0
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.84 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	7.0
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	18500
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	116.4
Beam depth	d_b (in)	24.5
Beam web thickness	t_{wb} (in)	0.605
Beam web area, $d_b t_{wb}$	A_w (in)	14.8225
Beam web width-to-thickness ratio, h / t_w	l_{web}	35.6
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb} A_w C_v$	ϕV_n (kips)	444.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.26 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb} F_y$)	$P_{d, total}$ (kips)	193.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	96.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	915.0
Beam flange thickness	t_{fb} (in)	0.96
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	8
Length of connection on beam	B4 (in)	25.25
Distance from beam centerline to bolt line	P2 (in)	2.75
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	7.104
Gross shear area, $2t_{fb} [B4]$	A_{gv} (in ²)	48.480
Net tension area, $A_{gt} - (t_{fb} (d_{hole, std} + 1/16))$	A_{nt} (in ²)	5.844
Net shear area, $A_{gv} - (t_{fb} (d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	29.580
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub} A_{nv} + F_{ub} A_{nt} < 0.6F_{yb} A_{gv} + F_{ub} A_{nt}$	R_n (kips)	1533.5
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1150.1
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.80 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

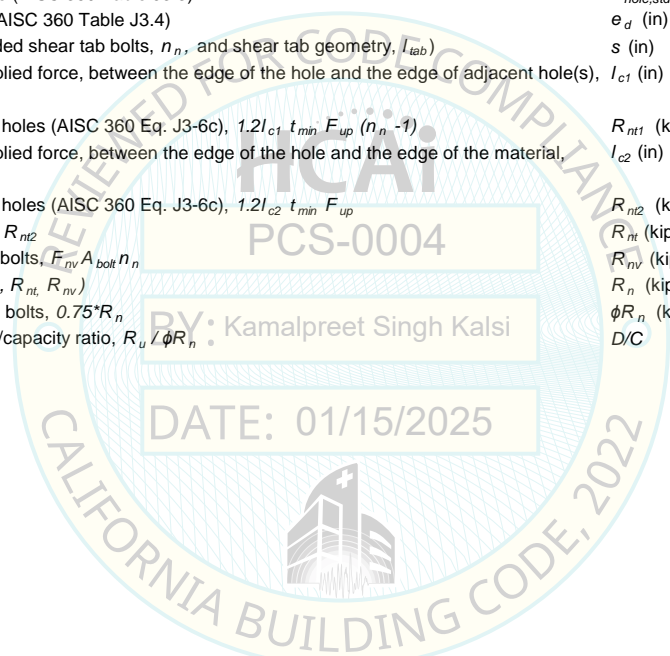
The required strength of shear tab bolt group (see step 10)	R_u (kips)	116.4
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.994
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	2
Provided number of shear tab bolts	n_n	4 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.605
Shear tab thickness	$T3$ (in)	0.5
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	351.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	477.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	34.1
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	511.9
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.46 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	116.4
Distance from alignment line to column	$C1$ (in)	6.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	712.8

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	6.88
Effective net area, $A_e = A_{nv}$	A_e (in ²)	6.88
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	201.09
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	335.16
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.35 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	116.37
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	712.77
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{ny} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nt} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nt})^2 + (V_u/\phi R_{ny})^4$	D/C	0.52 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.25
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	116.4
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	5.2
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	6.125
Size of weld in 1/16 ^{ths} of an inch	D_{w3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	712.8
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.3
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	9.0
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.81 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.5
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.77
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	457.5

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.125
Top plate dimension (see schedule)	$P6$ (in)	27.125
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	30.516
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	915.5
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	915.5
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.50 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	8
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	17.016
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	663.6
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	497.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.92 OK



Fig. C-15.8. Shear rupture of net section on top plate.

Note: For the alternate top plate, this check is conservative and does not need to be checked for the top plate.

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	457.5

Yielding check (LS 1)

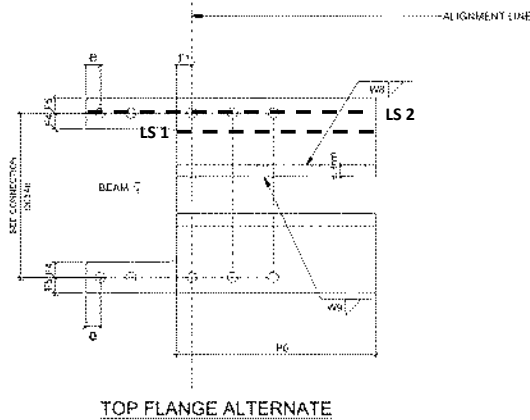
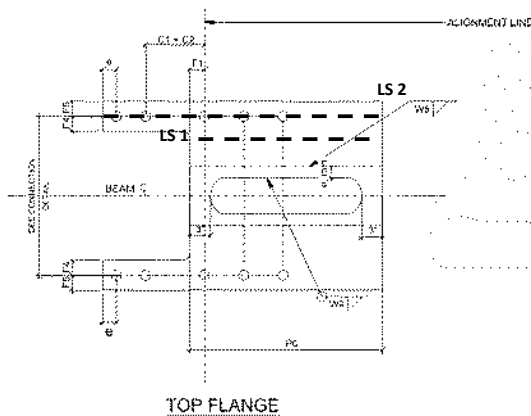
Top plate thickness (see step 2)	t_p (in)	1.125
Top plate dimension (see schedule)	$P6$ (in)	27.125
Fuse plate dimension	$F1$ (in)	1.75
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	30.516
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	915.5
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	915.5
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.50 OK

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Number of oversized bolts in Zone M (see step 5)	n_m	4
Number of oversized bolts on top plate (see step 5)	n_b	8
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Bolt spacing (see schedule)	s (in)	3.375
Fuse plate dimension	$F1$ (in)	1.75
Top plate dimension, $e_d + s*(n_m-1) + C1 + C2 + -F1 + P6$	$Dim 1$ (in)	42.25
Gross shear area of top plate, $(Dim 2)(t_p)$	A_g (in ²)	47.531
Number of oversized bolts on top plate (see step 5)	n_b	8
Net shear area of top plate, $A_g - (t_p d_o n_b)$	A_n (in ²)	34.594
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	1349.2
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	1011.9
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.45 OK

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u (kips)	457.5
Beam flange width	b_{fb} (in)	12.9
Weld 8 size	D_{W8} (1/16 in)	4
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.25
Weld 8 length, $P6 - 2l_{hb8}$	l_{w8} (in)	26.63
Weld 9 size	D_{W9} (1/16 in)	4
Weld 9 length, $l_{w8} - 2*3"$	l_{w9} (in)	20.63
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u / (l_{w8} + l_{w9})$	r_{uv} (kip/in)	9.7
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16 + D_{W9}/16)(1")$	ϕr_n (kip/in)	11.1
Alternative top plate-to-beam flange weld tearing check, $r_{uv} / \phi r_n$	D/C	0.87 OK



Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.5
Total number of bolts (see step 5)	n_b	8
Number of bolts in the M region (see step 5)	n_m	4
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	228.7

Yielding check

Top plate dimension	$P4$ (in)	1.875
Top plate dimension	$P5$ (in)	3.875
Top plate thickness (see step 2)	t_p (in)	1.125
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	6.469
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	323.4
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	291.1
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.79 OK

Rupture check

Top plate dimension	$P4$ (in)	1.875
Top plate dimension	$P5$ (in)	3.875
Top plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	4.781
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	310.8
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	233.1
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 OK

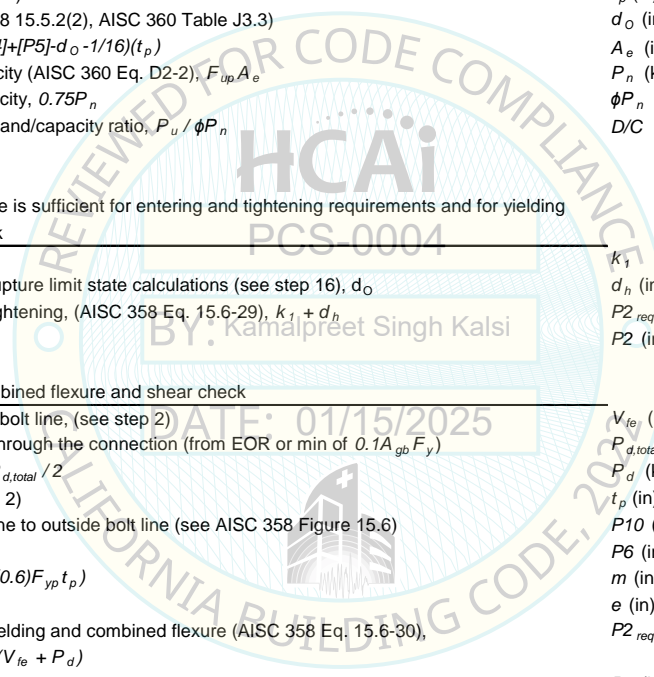
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.5
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.9375
Provided P2 dimension	$P2$ (in)	2.75 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.5
Top plate thickness (see step 2)	t_p (in)	1.125
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.075
Inside length of the top plate	$P6$ (in)	27.13
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	15.061
Dimension, $([P6] - m) / 2$	e (in)	6.032
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-4.0
Provided P2 dimension	$P2$ (in)	2.75 OK



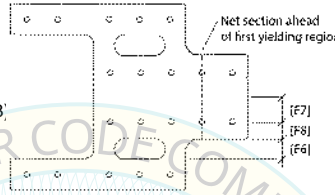
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.5 OK
Top plate thickness (see step 2)	t_p (in)	1.125
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.2 OK

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

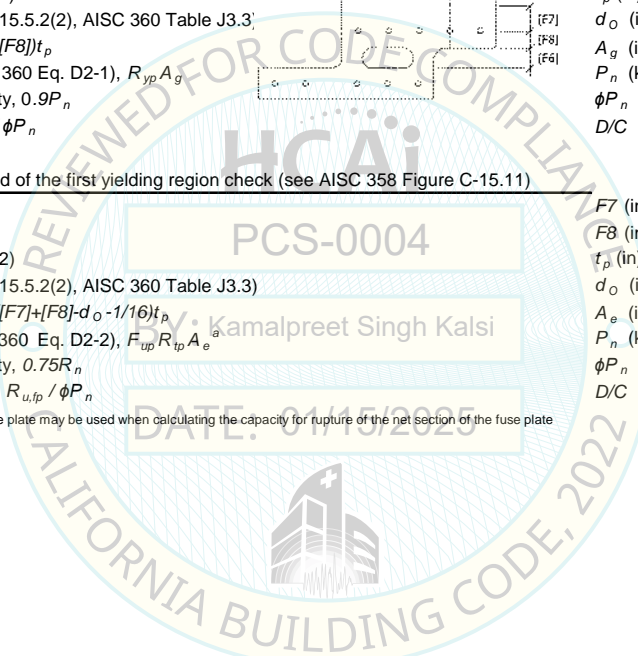
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	6
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	3
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	361.0
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	12.825
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	641.3
Factored tensile capacity, $0.9P_n$	ϕP_n (kips)	577.1
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.63 OK



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	9.450
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$	P_n (kips)	737.1
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	552.8
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.65 OK

*The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	361.0
Fuse plate thickness(see step 2)	t_p (in)	1.125
Width of fuse plate cut-out	$F6$ (in)	2.5
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B/[F6])/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.770
Provided yielding region depth	$F2$ (in)	2.750 OK
Fuse yield force (one side), $2(0.6F_{yp})/[F2]t_p$	V_y (kips)	185.6
Ratio	V_{fe} / V_y	1.9
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	0.91 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	6
Number of bolts in the M region	$n_{m, Bot}$	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	361.0
Yielding check		
Fuse plate dimension	$F4$ (in)	1.875
Fuse plate dimension	$F5$ (in)	2.375
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	9.5625
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	478.13
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	430.31
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	1.875
Fuse plate dimension	$F5$ (in)	2.375
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	6.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	482.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	362.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	96.5
External continuity plate dimension	C5 (in)	3
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	55.6
Cover plate overhang	C3 (in)	3
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.875
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕR_n (kips)	78.8
Tension failure demand/capacity ratio, $P_u/\phi R_n$	D/C	0.71 OK

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	55.6
Weld 5 size	D_{W5} (1/16 in)	7
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W5}/16([C3]-1)(2 \text{ sides})$	ϕR_n (kips)	58.5
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.95 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.5
Shear force from gravity	$V_{gravity}$ (kips)	7.0
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	18500
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	116.4
Distance from alignment line to column	C1 (in)	6.125
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	29.1
Weld 7 size	D_{W7} (1/16 in)	4
Cover plate overhang	C3 (in)	3
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_{W7}/16[C3](2 \text{ sides})$	ϕR_n (kips)	33.4
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.87 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	116.4
Distance from alignment line to column	C1 (in)	6.125
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	29.1
Weld 6 size	D_{W6} (1/16 in)	5
Cover plate overhang	C3 (in)	3
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	6
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W6}/16(l_{eff})$	ϕR_n (kips)	62.6
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.46 OK

Shear plate shear failure check (when $W1 > 3/4"$ without shear plates, shear plates are used)

Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Weld 4 size	D_{W4} (1/16 in)	7
Weld 4 effective length, $d_c - 2 t_{fc}$	l_{eff} (in)	19.78
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16(l_{eff})$	R_u (kips)	192.7
Shear plate thickness, $t_{sp} \geq 1.1W4 \text{ or } 3/8"$	t_{sp} (in)	0.500
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c - 2t_{fc})$	ϕR_n (kips)	296.7
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6 OK

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

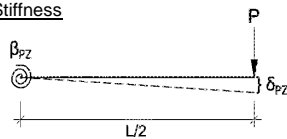
Show that effective panel zone stiffness $\geq 10 \times$ beam flexural stiffness

Effective Panel Zone Stiffness

$$K_{PZ} = P/\delta_{PZ}$$

$$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$$

$$K_{PZ} = \beta_{PZ}/(L/2)^2$$

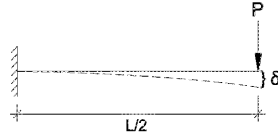


Beam Stiffness

$$K_B = P/\delta_B$$

$$\delta_B = P(L/2)^3/3EI$$

$$K_B = 3EI/(L/2)^3$$

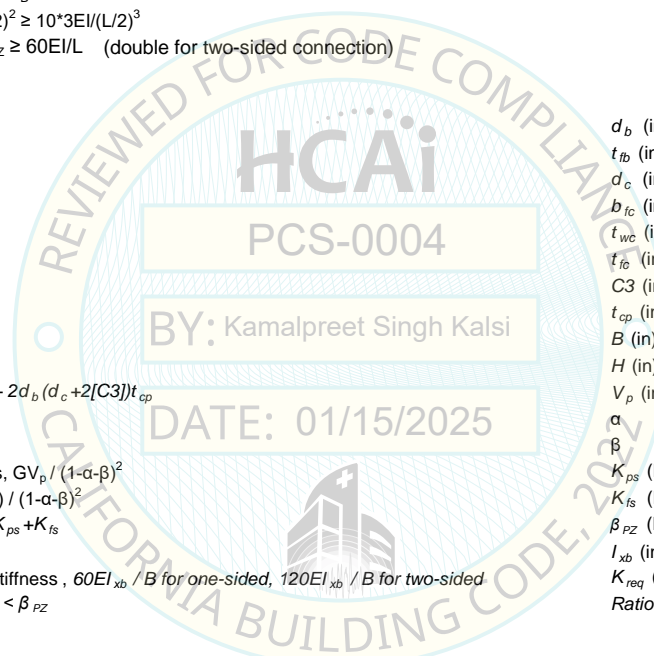


$$K_{PZ} \geq 10K_B$$

$$\beta_{PZ}/(L/2)^2 \geq 10 \times 3EI/(L/2)^3$$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.5
Beam flange thickness	t_{fb} (in)	0.96
Column depth	d_c (in)	21.7
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.6
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang	C3 (in)	3.0
Cover plate thickness	t_{cp} (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])t_{cp}$	V_p (in ³)	1506.6
Column ratio, $(d_c - t_{fc})/B$	α	0.058
Beam ratio, $(d_b - t_{fb})/H$	β	0.109
Web and cover plate stiffness, $GV_p/(1-\alpha-\beta)^2$	K_{ps} (k-in)	2.43E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2)/(1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.44E+07
Beam moment of inertia	I_{xb} (in ³)	4020
Required panel zone spring stiffness, $60EI_{xb}/B$ for one-sided, $120EI_{xb}/B$ for two-sided	K_{req} (k-in)	1.94E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	Ratio < 1	0.80 OK



Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)

In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	96.5
Total orthogonal drag load (from EOR)	$P_{d,do,total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total} / 2$	$P_{d,do}$ (kips)	0.0
Moment from orthogonal connection (seismic load combination) (from EOR)	$M_{u,o}$ (k-in)	391
Orthogonal beam size		W24X55
Orthogonal beam depth	$d_{b,o}$ (in)	23.6
Orthogonal beam flange thickness	$t_{fb,o}$ (in)	0.51
Flange force associated with $M_{u,o}$, $M_{u,o} / (d_{b,o} - t_{fb,o})$	$P_{d,mo}$ (kips)	16.9
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	16.9
Column flange width	b_{fc} (in.)	12.4
Column depth	d_c (in.)	21.7
Column flange thickness	t_{fc} (in.)	0.96
Column web thickness	t_{wc} (in.)	0.60
Plate thickness (see T2 in drawings)	t_p (in)	1.13
External continuity plate gage dimension	C5 (in.)	3.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d) / (1 + 0.5t_{wc} / t_p)]$	R_{cp} (kips)	361.2
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	96.3
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	16.9

External continuity plate section check (at the coverplate)

Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	1372.4
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	16.9
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	457.5
Plate plastic section modulus, $(d_c - 2t_{fc})^2 t_p / 4$	Z (in ³)	110.04
Plate section area, $(d_c - 2t_{fc}) t_p$	A (in ²)	22.25
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	4951.7
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	1001.4
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	650.9
Interaction equation, $(M_r / M_c) + (P_r / P_c)^2 + (V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.52 OK
		Special detailing not required

External continuity plate section check (at the column web)

Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5]) - R_{cp}(b_{fc}/2)$	M_r (k-in)	1969.6
Normal demand at section, $=P_{d,o}$	P_r (kips)	16.9
Shear demand at section, $=R_{w1}$	V_r (kips)	96.3
Corner clip dimension	K_c (in)	1.46
Plate plastic section modulus, $(d_c - 2t_{fc} - 2K_c)^2 t_p / 4$	Z (in ³)	79.9
Plate section area, $(d_c - 2t_{fc} - 2K_c)t_p$	A (in ²)	18.97
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	3597.7
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	853.5
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	554.8
Interaction equation, $(M_r / M_c) + (P_r / P_c)^2 + (V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.55 OK
		Special detailing not required

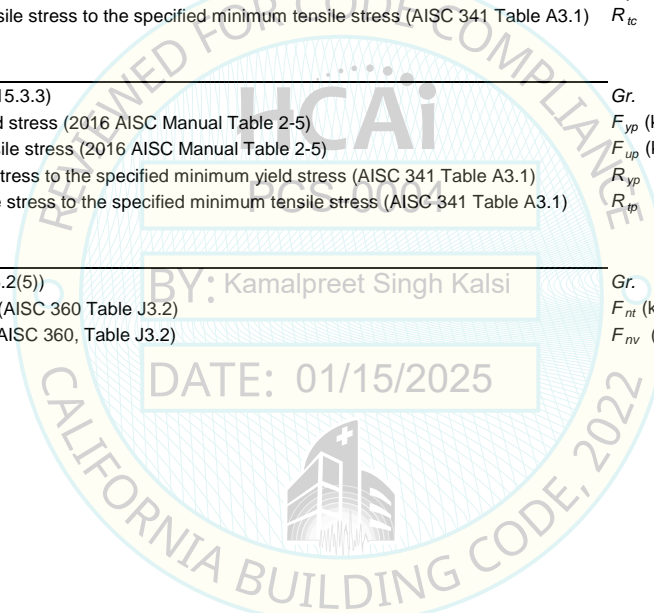
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF153
W40X324 Column and W40X431 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



DATE: 01/15/2025

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W40X431
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	41.3 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	431
Beam weight	W_b (lb/ft)	431 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	25.5 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_yb)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	3.44 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))	See bracing calculations	

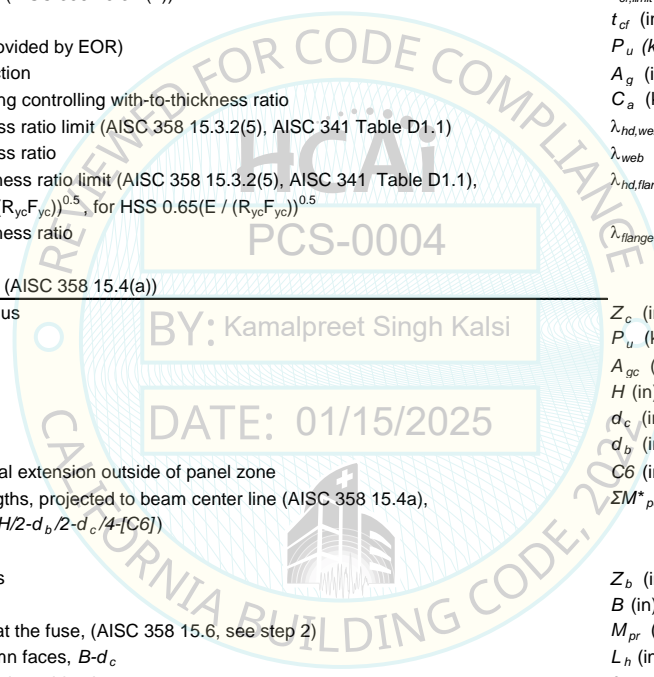
Column limitation checks

		W40X324
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	40.2
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	324 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	1.81 OK
Axial force on the column (provided by EOR)	P_u (kips)	270.9
Gross area of the column section	A_g (in ²)	95.3
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.1
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	55.5
Column web width-to-thickness ratio	λ_{web}	34.2 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	4.4 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	1460
Column design axial load	P_u (kips)	270.9
Column area	A_{gc} (in ²)	95.3
Story Height	H (in)	216.0
Column depth	d_c (in)	40.2
Beam depth	d_b (in)	41.3
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	2.00E+05
Beam plastic section modulus	Z_b (in ³)	1960
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	98000
Clear distance between column faces, $B - d_c$	L_h (in)	319.8
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	43.8
Beam shear (AISC 358 15.4a), $2M_{pr} / L_h + V_g$	V_b (kips)	656.7
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	13200
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	1.11E+05
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	Ratio > 1.0	1.80 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1))



Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	8404
Beam plastic section modulus	Z_b (in ³)	1960
Beam plastic bending moment, $F_y Z_b$	M_p (k-in)	98000
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	98000 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	2
Beam depth	d_b (in)	41.3
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	1131.6
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	1131.6

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1131.6
Beam area	A_{gb} (in ²)	127.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	317.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	1449.1
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	91
Plate thickness, t_p (see T2 on drawings)	t_p (in)	2
Beam depth	d_b (in)	41.3
Column depth	d_c (in)	40.2
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	1435.3
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	1.25
Cover plate overhang dimension	C3 (in)	9.25
Column depth	d_c (in)	40.2
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	58.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	2201.3
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	2201.3
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.66 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	47.30
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	1773.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	1773.8
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.81 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	41.3
Beam flange thickness	t_{fb} (in)	2.36
Column flange thickness	t_{fc} (in)	1.81
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	36.58
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	36.58
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.813
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.54 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.25
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.375
Beam plastic section modulus	Z_b (in ³)	1960
Beam flange thickness	t_{fb} (in)	2.36
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	3.540
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	275.7
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	1684.3
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	107800
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	120428
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.90 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	635.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	317.5
Bolt diameter	d_{bolt} (in)	1.25
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in ²)	1.227
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	103.1
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$	$n_{b, req}$	14.1
Number of bolts per line (AISC 358 15.2(1))	n_b	19 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.75
Bolt spacing (see schedule)	s (in)	3.75
Plate thickness, t_p (see T2 in drawings)	t_p (in)	2
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	6.045
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a] + 3in - 2e_d] / s + 1$	$n_{p, min}$	4.1
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	10 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	9

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1449.1
Beam flange thickness	t_{fb} (in)	2.36
Fuse and top plate thickness	$T2$ (in)	2
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	2.0
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	7410.0
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.5625
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	2.2
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	6142.5
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o / 2$	l_{c2} (in)	1.0
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	151.1
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	6293.6
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1958.6
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1958.6
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1468.9
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.99 OK

Bolt slip for wind load check

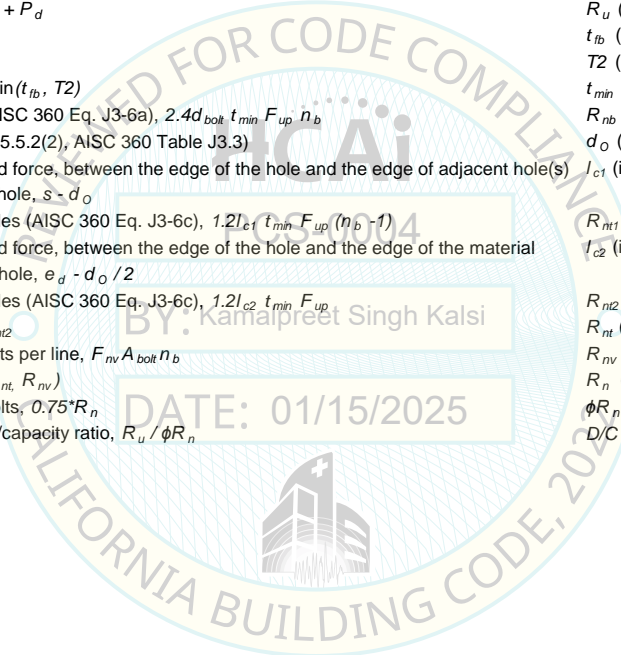
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068
Bolt group demand from wind per line, $M_u / (2(d_b + t_p))$	R_u (kip)	35.4
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	19
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	657.0
Total factored bolt slip resistance, $0.85 R_n$	ϕR_n (kips)	558.4
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.06 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1131.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0
Bolt diameter	d_{bolt} (in)	1.25
Area of single bolt, $\pi*d_{bolt}^2/4$	A_{bolt} (in ²)	1.227
Bolt shear strength, $\phi F_{nv} A_b$	ϕR_n (kips)	103.1
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi R_n$	$n_{b, req}$	11.0
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b, Bot}$	15 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.75
Bolt spacing (see schedule)	s (in)	3.75
Plate thickness, t_p (see T2 in drawings)	t_p (in)	2
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	6.045
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	4.1
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p, Bot}$	8 OK
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), $nb - np$	$n_{m, Bot}$	7

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1131.6
Beam flange thickness	t_{fb} (in)	2.36
Fuse and top plate thickness	$T2$ (in)	2
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	2.0
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	7410.0
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.5625
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) I_{c1} is based on an oversized hole, $s - d_o$	I_{c1} (in)	2.2
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	4777.5
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material I_{c2} is based on an oversized hole, $e_d - d_o / 2$	I_{c2} (in)	1.0
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c2} t_{min} F_{up}$	R_{nt2} (kips)	151.1
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	4928.6
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1546.3
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1546.3
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	1159.7
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.98 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	41.3
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.75
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	1
Cover plate overhang dimension	C3 (in)	9.25
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max}d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	13.478
Distance from alignment line to column	C1 (in)	13.5 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	317.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	91
Column depth	d_c (in)	40.2
Column flange thickness	t_{fc} (in)	1.81
Beam depth	d_b (in)	41.3
Plate thickness (see T2 on drawings)	t_p (in)	2
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	1.25
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	47.3
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	4.75
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.6875
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	45.93
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	6.25
Weld 1 size	D_{W1} (1/16 in)	9
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	9
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	36.58
Horizontal distance from center of rotation to geometric center	e_x (in)	1.37
Vertical distance from center of rotation to geometric center	e_y (in)	4.37

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) \cdot L$ ΣR_L (k-in) 76449

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R \cdot L_R$ (k-in)
W1RT	32.76	20.71	55.12	678.5
W4TL	33.81	14.82	34.79	501.0
W1LB	28.10	19.32	40.19	543.1
W4BR	25.23	15.40	41.02	388.4
W1RC	19.34	14.89	14.45	288.1
W4TC	27.82	11.29	3.57	314.1
W1LC	22.00	15.01	12.67	330.2
W4BC	19.11	10.65	5.20	203.6
		20.71	55.12	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	20.7
Orientation of Resultant Force	θ_r (degrees)	55.1
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.92
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma R_L)$	r_{uv} (kip/in)	16.5
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c \cdot l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	16.5
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.30
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 \cdot 0.6 \cdot 70 \cdot (1 + 0.5 \cdot \sin^{1.5}(\theta_r)) \cdot (p_i \cdot (1.9 - 0.9p_i))^{0.3} \cdot 0.707 \cdot D_{W1}/16)$	ϕr_n (kips)	16.91
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.97 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	317.5
Column depth	d_c (in)	40.2
Column flange thickness	t_{fc} (in)	1.81
Cover plate overhang dimension	C3 (in)	9.25
External continuity plate dimension	C5 (in)	4.75
Weld 2 size	D_{W2} (1/16 in)	17
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	1.06
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	11.25
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	24.7
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3]) - 2l_{hb2} - l_{we2,eff})$	r_{un2} (kip/in)	13.5
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	28.1
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2 \text{ sides})$	ϕR_n (kip/in)	47.3
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.59 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1131.6
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	6883.4
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	1449.1
External continuity plate thickness (see step 2)	t_p (in)	2.000
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c + 2[C3])t_p$	ϕR_n (kips)	3522.0
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	77528.0
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.50 OK

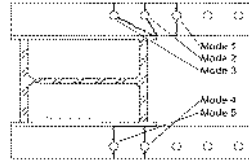
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	317.5
Column fillet dimension	k (in)	3.0
Column fillet dimension	$k1$ (in)	1.7
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	181.6
Column flange width	b_{fc} (in)	15.9
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	56.85
Weld 2 size	D_{W2} (1/16 in)	17
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	5729.1
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	1793.6
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	7560.2
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	5670.1
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.26 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	635.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	317.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	10.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	762.7
External continuity plate thickness (see step 2)	t_p (in)	2
External continuity plate width	$C4$ (in)	10
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Net area of external continuity plate cross section, $t_p ([C4] - (d_{hole,std} + 1/16))$	A_n (in ²)	17.125
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	1113.1
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	834.8
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.91 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

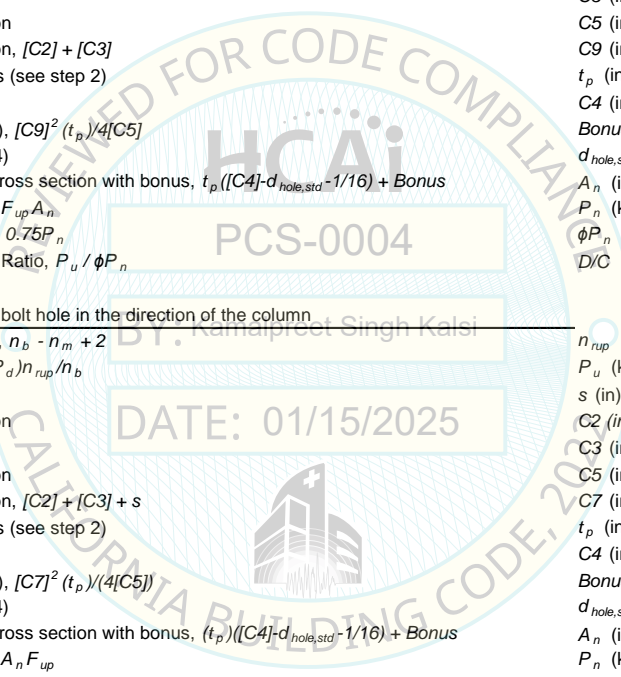
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	11
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	839.0
External continuity plate dimension	$C2$ (in)	-7.875
Cover plate overhang dimension	$C3$ (in)	9.25
External continuity plate dimension	$C5$ (in)	4.75
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	1.375
External continuity plate thickness (see step 2)	t_p (in)	2
External continuity plate width	$C4$ (in)	10
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p) / (4[C5])$	$Bonus$ (in)	0.199
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Area of external continuity plate cross section with bonus, $t_p ([C4] - d_{hole,std} - 1/16) + Bonus$	A_n (in ²)	17.324
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	1126.1
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	844.5
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.99 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	12
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	915.2
Bolt spacing (see schedule)	s (in)	3.75
External continuity plate dimension	$C2$ (in)	-7.875
Cover plate overhang dimension	$C3$ (in)	9.25
External continuity plate dimension	$C5$ (in)	4.75
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.125
External continuity plate thickness (see step 2)	t_p (in)	2
External continuity plate width	$C4$ (in)	10
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p) / (4[C5])$	$Bonus$ (in)	2.765
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Area of external continuity plate cross section with bonus, $(t_p) ([C4] - d_{hole,std} - 1/16) + Bonus$	A_n (in ²)	19.890
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	1292.8
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	969.6
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.94 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	11
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	839.0
External continuity plate dimension	$C2$ (in)	-7.875
Cover plate overhang dimension	$C3$ (in)	9.25
External continuity plate width	$C4$ (in)	10
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	1.375
External continuity plate thickness (see step 2)	t_p (in)	2
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	20.000
Gross shear area, $t_p [C9]$	A_{gv} (in ²)	2.750
Net tension area, $A_{gt} - ((t_p) * (d_{hole,std} + 1/16))$	A_{nt} (in ²)	17.125
Net shear area, A_{gv}	A_{nv} (in ²)	2.750
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	1195.6
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	896.7
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.94 OK



Step 9. (continued)

<u>Mode 5: Block shear through the second bolt hole in the direction of the column</u>		
Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	12
Mode 5 rupture demand, $(V_{fe} + P_d) \cdot n_{rup} / n_b$	R_u (kips)	915.2
Bolt spacing (see schedule)	s (in)	3.75
External continuity plate dimension	C2 (in)	-7.875
Cover plate overhang dimension	C3 (in)	9.25
External continuity plate width	C4 (in)	10
External continuity plate dimension, $[C2] + [C3] + s$	C7 (in)	5.125
External continuity plate thickness (see step 2)	t_p (in)	2
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.375
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	20.000
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	10.250
Net tension area, $A_{gt} - (t_p (d_{hole} + 1/16))$	A_{nt} (in ²)	17.125
Net shear area, A_{gv}	A_{nv} (in ²)	10.250
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	1420.6
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	1065.5
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.86 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	43.8
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	98000
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	656.7
Beam depth	d_b (in)	41.3
Beam web thickness	t_{wb} (in)	1.34
Beam web area, $d_b t_{wb}$	A_w (in)	55.342
Beam web width-to-thickness ratio, h / t_w	l_{web}	25.5
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_wC_v$	ϕV_n (kips)	1660.3
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.40 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	635.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	317.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	2898.3
Beam flange thickness	t_{fb} (in)	2.36
Beam flange width	b_{fb} (in)	16.2
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.375
Number of holes along each bolt line (see step 5)	n_b	19
Length of connection on beam	B4 (in)	69.25
Distance from beam centerline to bolt line	P2 (in)	6.4375
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	7.847
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	326.860
Net tension area, $A_{gt} - (t_{fb} (d_{hole, std} + 1/16))$	A_{nt} (in ²)	4.455
Net shear area, $A_{gv} - (t_{fb} (d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	201.338
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	8141.7
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	6106.3
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.47 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

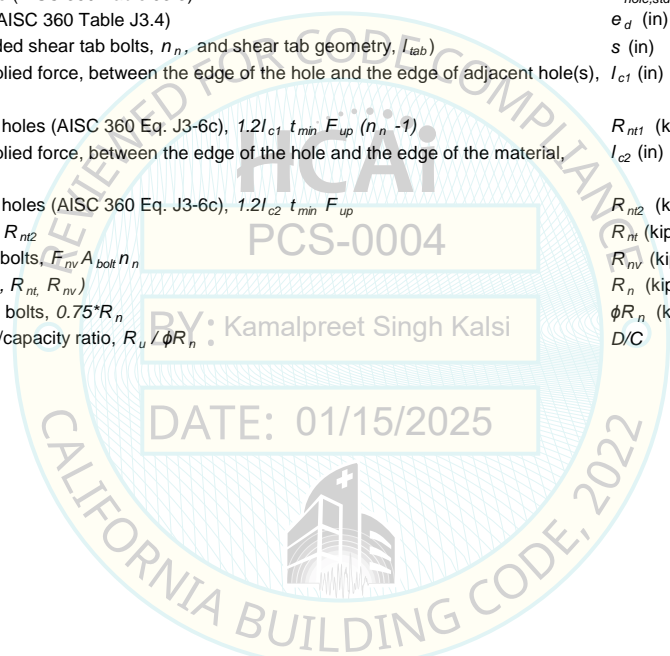
The required strength of shear tab bolt group (see step 10)	R_u (kips)	656.7
Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	1.227
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	103.1
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	77.3
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	8
Provided number of shear tab bolts	n_n	9 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	34
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	33
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	33 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	1.34
Shear tab thickness	$T3$ (in)	1.25
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	1.3
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	2193.8
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	3.719
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	2.3
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	1828.1
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	91.4
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1919.5
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	927.8
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	927.8
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	695.8
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.94 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	102
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	51.9
Provided number of shear tab bolts (see step 12)	n_n	9
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	466.8
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	656.7
Distance from alignment line to column	$C1$ (in)	13.5
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	8865.6

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Slotted bolt hole width (standard slot width)	w_b (in)	1.375
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	1.25
Shear tab length (see step 12)	l_{tab} (in)	33
Beam T dimension limit	T_{beam} (in)	34
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	3.6875
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	41.25
Shear tab net area, $A_{gv} - n_n (w_b + 1/16)t_{tab}$	A_{nv} (in ²)	25.08
Effective net area, $A_e = A_{nv}$	A_e (in ²)	25.08
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up}A_{nv}$	ϕR_{nv} (kips)	733.54
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up}A_e$	ϕR_{tn} (kips)	1222.56
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.79 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	656.71
Required axial strength of the shear tab (see above)	P_u (kips)	466.80
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	8865.63
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	41.25
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	340.31
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp}A_{gv}$	ϕR_{ny} (kips)	1237.50
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp}A_{gv}$	ϕR_{nt} (kips)	1856.25
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	15314.06
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nt})^2 + (V_u/\phi R_{ny})^4$	D/C	0.72 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	37.3
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	656.7
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	17.6
Number of slotted bolt holes in shear tab	n_{vb}	9
Slotted bolt hole spacing	s_{slot} (in)	3.6875
Distance from line of bolts to shear tab weld	$C1$ (in)	13.5
Size of weld in 1/16 ^{ths} of an inch	D_{w3}	8
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	8865.6
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	12.5
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	21.6
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	22.3
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.97 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.75
Beam depth	d_b (in)	41.3
Length of the shear tab (see step 12)	l_{tab} (in)	33
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	2.749
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	2.75 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	2.75

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	317.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1449.1

Yielding check

Top plate thickness (see step 2)	t_p (in)	2
Top plate dimension (see schedule)	$P6$ (in)	71.5
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	143.000
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	4290.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	4290.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.34 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.625
Number of short slotted bolts on top plate (see step 5)	n_b	19
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	81.250
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	3168.8
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	2376.6
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.61 OK



Fig. C-15.8. Shear rupture of net section on top plate.

Note: For the alternate top plate, this check is conservative and does not need to be checked for the top plate.

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	317.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1449.1

Yielding check (LS 1)

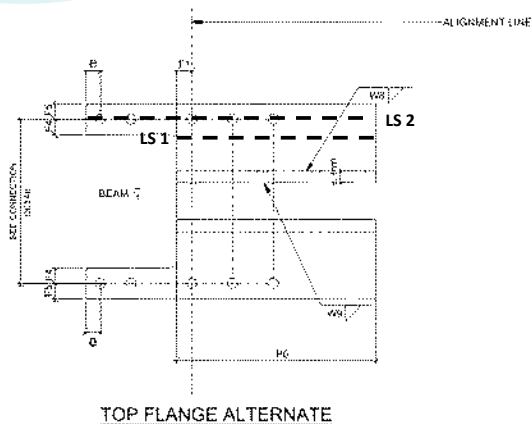
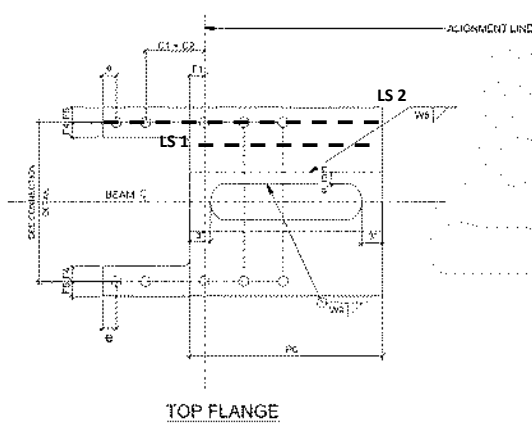
Top plate thickness (see step 2)	t_p (in)	2
Top plate dimension (see schedule)	$P6$ (in)	71.5
Fuse plate dimension	$F1$ (in)	2.00
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	143.000
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	4290.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	4290.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.34 OK

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.5625
Number of oversized bolts in Zone M (see step 5)	n_m	9
Number of oversized bolts on top plate (see step 5)	n_b	19
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.75
Fuse plate dimension	$F1$ (in)	2.00
Top plate dimension, $e_d + s*(n_m-1) + C1 + C2 + -F1 + P6$	$Dim 1$ (in)	106.75
Gross shear area of top plate, $(Dim 2)(t_p)$	A_g (in ²)	213.500
Number of oversized bolts on top plate (see step 5)	n_b	19
Net shear area of top plate, $A_g - (t_p d_o n_b)$	A_n (in ²)	154.125
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	6010.9
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	4508.2
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.32 OK

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u (kips)	1449.1
Beam flange width	b_{fb} (in)	16.2
Weld 8 size	D_{W8} (1/16 in)	4
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.25
Weld 8 length, $P6 - 2l_{hb8}$	l_{w8} (in)	71.00
Weld 9 size	D_{W9} (1/16 in)	4
Weld 9 length, $l_{w9} - 2*3"$	l_{w9} (in)	65.00
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u / (l_{w8} + l_{w9})$	r_{uv} (kip/in)	10.7
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16 + D_{W9}/16)(1")$	ϕr_n (kip/in)	11.1
Alternative top plate-to-beam flange weld tearing check, $r_{uv} / \phi r_n$	D/C	0.96 OK



Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	317.5
Total number of bolts (see step 5)	n_b	19
Number of bolts in the M region (see step 5)	n_m	9
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	686.4

Yielding check

Top plate dimension	P4 (in)	2.25
Top plate dimension	P5 (in)	6.5
Top plate thickness (see step 2)	t_p (in)	2
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	17.500
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	875.0
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	787.5
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.87 OK

Rupture check

Top plate dimension	P4 (in)	2.25
Top plate dimension	P5 (in)	6.5
Top plate thickness (see step 2)	t_p (in)	2
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.5625
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	14.250
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	926.3
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	694.7
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99 OK

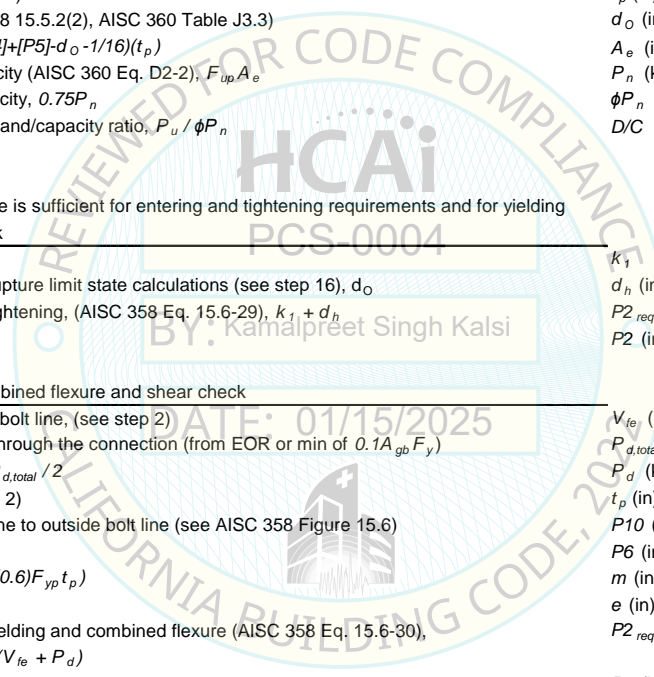
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.9
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.5625
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3.4375
Provided P2 dimension	P2 (in)	6.4375 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	317.5
Top plate thickness (see step 2)	t_p (in)	2
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	P10 (in)	13.95
Inside length of the top plate	P6 (in)	71.50
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	26.836
Dimension, $([P6] - m) / 2$	e (in)	22.332
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-54.2
Provided P2 dimension	P2 (in)	6.4375 OK



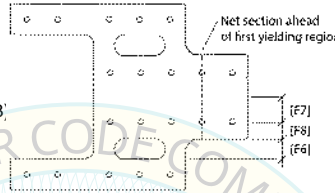
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	3.5 OK
Top plate thickness (see step 2)	t_p (in)	2
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	1.8 OK

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

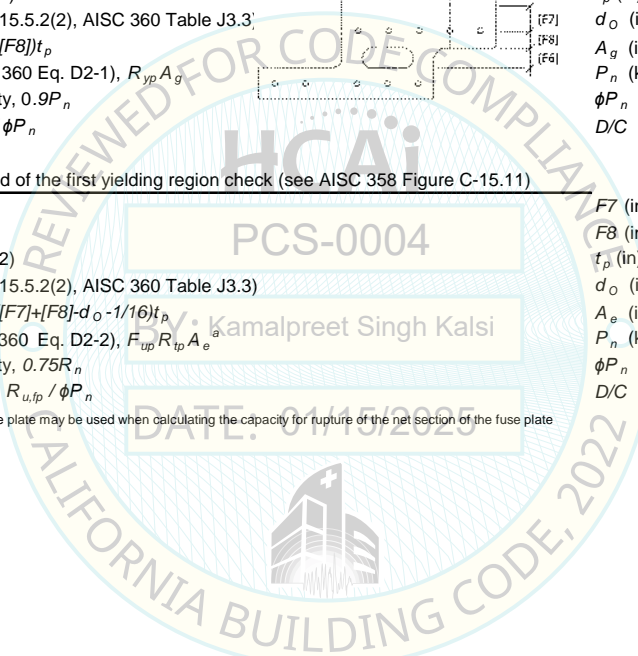
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	15
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	7
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	1056.2
Fuse plate dimension, centerline to gage line	$F7$ (in)	6.4375
Fuse plate dimension, gage line to outside edge	$F8$ (in)	1.7625
Fuse plate thickness (see step 2)	t_p (in)	2
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.5625
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	32.800
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	1640.0
Factored tensile capacity, $0.9P_n$	ϕP_n (kips)	1476.0
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.72 OK



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	6.4375
Fuse plate dimension	$F8$ (in)	1.7625
Fuse plate thickness (see step 2)	t_p (in)	2
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.563
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	26.300
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$	P_n (kips)	2051.4
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	1538.6
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.69 OK

*The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Step 19. Determine the yielding region depth [F2]

Applicable to AISC 358-27

Width of fuse plate cut-out	F6 (in)	3.5
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	98000
Fuse plate thickness(see step 2)	t_p (in)	2
Beam depth	d_b (in)	41.3
Strain hardening factor, $R_{tp} F_{up}/R_y F_{yp}$	C_{pr}	1.42
Beam shear demand (see step 10), $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	656.7
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_n	9
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	311.2
Resistance factor for bolt slip	ϕ	0.7
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	326.8
Shear tab bolt slip force multiplied by 0.4, $\phi R_n * 0.4$	R_{tab} (kips)	130.7
Maximum yielding region depth (AISC 358-27 Eq. 15.6-34), $(0.9M_{pr}-R_{tab}(d_b+t_p)/2)/((d_b+t_p)*4t_p(0.6R_{yp}F_{yp}C_{pr}))$	F2 max (in)	5.266
Provided yielding region depth	F2 (in)	5.250 OK
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.85$	F6 / F2	0.66 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	15
Number of bolts in the M region	$n_{m, Bot}$	7
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	1056.2
Yielding check		
Fuse plate dimension	F4 (in)	2.25
Fuse plate dimension	F5 (in)	4
Fuse plate thickness (see step 2)	t_p (in)	2
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.5625
Gross area of yielding, $2((F4)+(F5))(t_p)$	A_g (in ²)	25
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	1250.00
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	1125.00
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	F4 (in)	2.25
Fuse plate dimension	F5 (in)	4
Fuse plate thickness (see step 2)	t_p (in)	2
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.5625
Effective net area of rupture, $2((F4)+(F5)-d_o-1/16)(t_p)$	A_e (in ²)	18.5
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	1443.0
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	1082.3
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.98 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1131.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	635.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	317.5
External continuity plate dimension	C5 (in)	4.75
Column depth	d_c (in)	40.2
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	139.2
Cover plate overhang	C3 (in)	9.25
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕR_n (kips)	232.0
Tension failure demand/capacity ratio, $P_u/\phi R_n$	D/C	0.60 OK

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	139.2
Weld 5 size	D_{W5} (1/16 in)	5
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W5}/16([C3]-1)(2 \text{ sides})$	ϕR_n (kips)	172.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.81 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	40.2
Beam depth	d_b (in)	41.3
Shear force from gravity	$V_{gravity}$ (kips)	43.8
Maximum probable moment at fuse location (see Step 2)	M_{pr} (K-in)	98000
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	656.7
Distance from alignment line to column	C1 (in)	13.5
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	214.7
Weld 7 size	D_{W7} (1/16 in)	9
Cover plate overhang	C3 (in)	9.25
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_{W7}/16[C3](2 \text{ sides})$	ϕR_n (kips)	231.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.93 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	656.7
Distance from alignment line to column	C1 (in)	13.5
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	214.7
Weld 6 size	D_{W6} (1/16 in)	8
Cover plate overhang	C3 (in)	9.25
Column flange width	b_{fc} (in)	15.9
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	13.9
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W6}/16(l_{eff})$	ϕR_n (kips)	232.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.92 OK

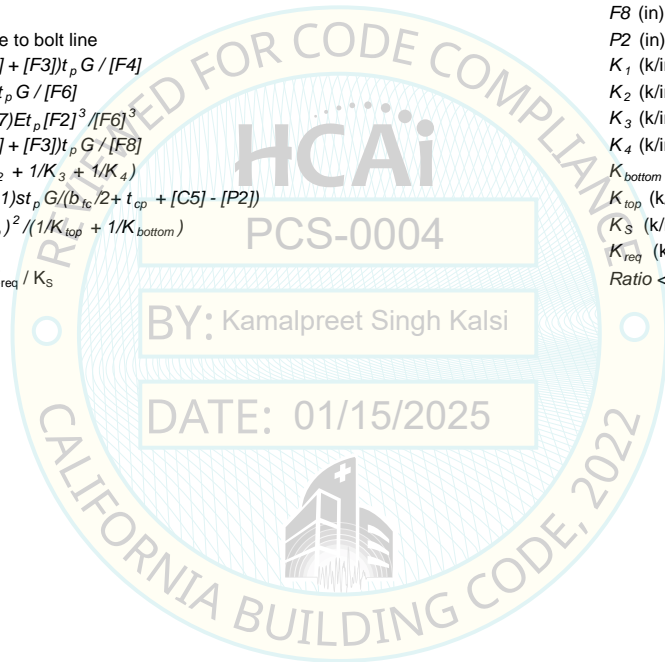
Shear plate shear failure check (when $W1 > 3/4"$ without shear plates, shear plates are used)

Column depth	d_c (in)	40.2
Column flange thickness	t_{fc} (in)	1.81
Weld 4 size	D_{W4} (1/16 in)	9
Weld 4 effective length, $d_c - 2 t_{fc}$	l_{eff} (in)	36.58
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16(l_{eff})$	R_u (kips)	458.2
Shear plate thickness, $t_{sp} \geq 1.1W4 \text{ or } 3/8"$	t_{sp} (in)	0.625
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c - 2t_{fc})$	ϕR_n (kips)	685.9
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.7 OK

Connection stiffness check

The purpose of this check is to ensure that the connection has enough stiffness to be considered Fully Restrained (FR) when M_{pr} is less than M_p (see AISC 358-22 15.6 Step 2). Moment frame connections are considered FR for design purposes if the connection stiffness is large relative to the beam flexural stiffness (EI/L). A threshold of $18EI/L$ is used in Chapter 13 of AISC 358 based on experimental testing. This procedure follows the method in the commentary to AISC 358-22 15.6 Step 2.

Beam moment of inertia	I_{xb} (in ³)	34800
Column flange width	b_{fc} (in)	15.9
Beam depth	d_b (in)	41.3
Bay width	B (in)	360.0
Number of bolts in zone M	M	9
Number of bolts in zone P	P	10
Bolt spacing	s (in)	3.75
Cover plate thickness	t_{cp} (in)	1.25
Fuse plate thickness	t_p (in)	2
External continuity plate dimension	C5 (in)	4.75
Fuse plate dimension	F2 (in)	5.25
Fuse plate dimension	F3 (in)	19.5
Fuse plate dimension	F4 (in)	2.3
Fuse plate dimension	F6 (in)	3.5
Fuse plate dimension	F8 (in)	1.7625
Distance from beam centerline to bolt line	P2 (in)	6.4375
Fuse region 1 stiffness, $(2[F2] + [F3])t_p G / [F4]$	K_1 (k/in)	2.99E+05
Fuse region 2 stiffness, $2[F2]t_p G / [F6]$	K_2 (k/in)	6.72E+04
Fuse region 3 stiffness, $2(0.37)Et_p [F2]^3 / [F6]^3$	K_3 (k/in)	1.45E+05
Fuse region 4 stiffness, $(2[F2] + [F3])t_p G / [F8]$	K_4 (k/in)	3.81E+05
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	7.21E+04
Top plate stiffness, $(2P + M - 1)st_p G / (b_{fc}/2 + t_{cp} + [C5] - [P2])$	K_{top} (k/in)	3.13E+05
Connection stiffness, $(d_b + t_p)^2 / (1/K_{top} + 1/K_{bottom})$	K_S (k/in)	N/A
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	N/A
Connection stiffness check, K_{req} / K_S	Ratio < 1	N/A N/A



Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

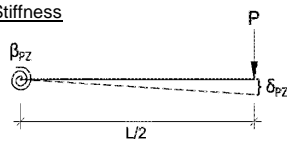
Show that effective panel zone stiffness $\geq 10 \times$ beam flexural stiffness

Effective Panel Zone Stiffness

$$K_{PZ} = P/\delta_{PZ}$$

$$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$$

$$K_{PZ} = \beta_{PZ}/(L/2)^2$$

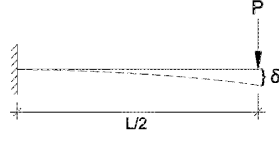


Beam Stiffness

$$K_B = P/\delta_B$$

$$\delta_B = P(L/2)^3/3EI$$

$$K_B = 3EI/(L/2)^3$$

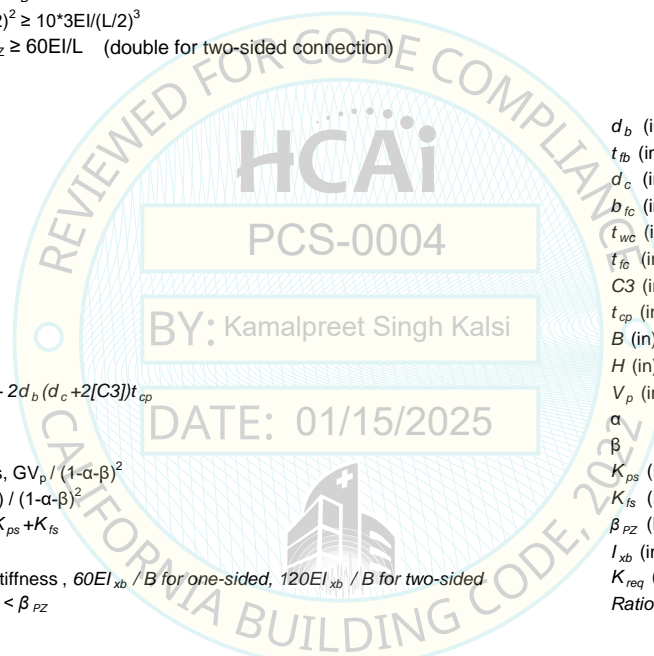


$$K_{PZ} \geq 10K_B$$

$$\beta_{PZ}/(L/2)^2 \geq 10 \times 3EI/(L/2)^3$$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	41.3
Beam flange thickness	t_{fb} (in)	2.36
Column depth	d_c (in)	40.2
Column flange width	b_{fc} (in)	15.9
Column web thickness	t_{wc} (in)	1
Column flange thickness	t_{fc} (in)	1.81
Cover plate overhang	C3 (in)	9.3
Cover plate thickness	t_{cp} (in)	1.25
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])t_{cp}$	V_p (in ³)	7721.0
Column ratio, $(d_c - t_{fc})/B$	α	0.107
Beam ratio, $(d_b - t_{fb})/H$	β	0.180
Web and cover plate stiffness, $GV_p/(1-\alpha-\beta)^2$	K_{ps} (k-in)	1.70E+08
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2)/(1-\alpha-\beta)^2$	K_{fs} (k-in)	8.95E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	1.71E+08
Beam moment of inertia	I_{xb} (in ³)	34800
Required panel zone spring stiffness, $60EI_{xb}/B$ for one-sided, $120EI_{xb}/B$ for two-sided	K_{req} (k-in)	1.68E+08
Rigid panel zone check, $K_{req} < \beta_{PZ}$	Ratio < 1	0.98 OK



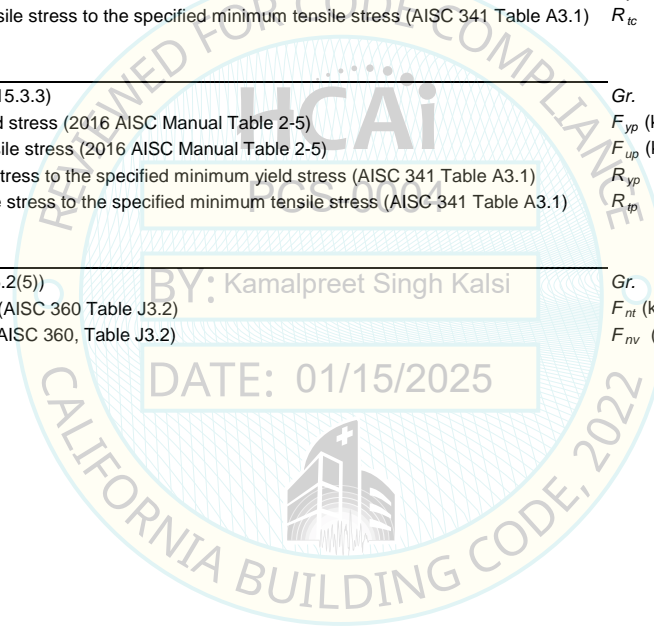
DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF250
W21X122 Column and W24X94 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200
<hr/>		
Beams		
Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1
<hr/>		
Columns		
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1
<hr/>		
Plates		
Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2
<hr/>		
Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W24X94
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	24.3 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	94 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	41.9 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_y)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	5.18 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))	See bracing calculations	

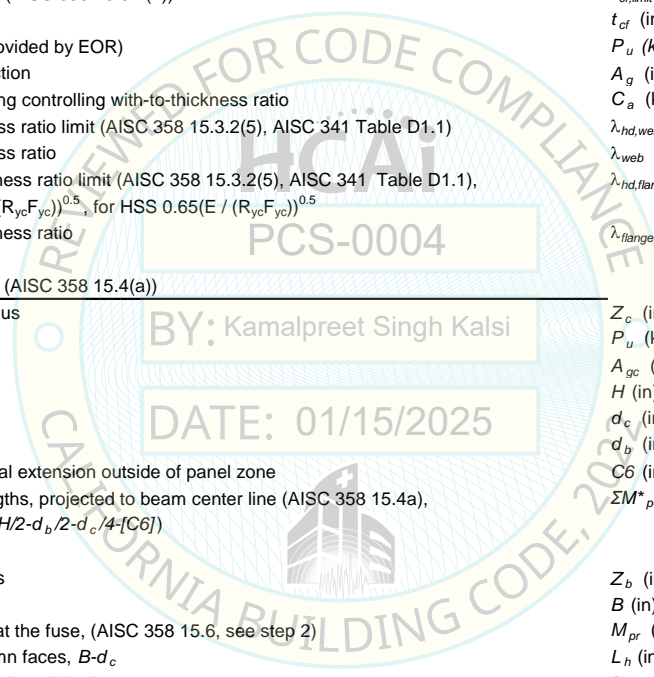
Column limitation checks

		W21X122
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	21.7
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	122 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.96 OK
Axial force on the column (provided by EOR)	P_u (kips)	53.5
Gross area of the column section	A_g (in ²)	35.9
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.0
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	57.2
Column web width-to-thickness ratio	λ_{web}	31.3 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	6.45 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	307
Column design axial load	P_u (kips)	53.5
Column area	A_{gc} (in ²)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.3
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\sum Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	$\sum M_{pc}^*$ (k-in)	N/A
Beam plastic section modulus	Z_b (in ³)	254
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	9535
Clear distance between column faces, $B - d_c$	L_h (in)	338.3
Live load factor in seismic load combination	f_l	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f_lL + 0.2S$	V_g (kips)	7.0
Beam shear (AISC 358 15.4a), $2M_{pr} / L_h + V_g$	V_b (kips)	63.4
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	688
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\sum(M_{pr} + M_{uv})$	$\sum M_{pb}^*$ (k-in)	N/A
Column-beam moment ratio, $\sum M_{pc}^* / \sum M_{pb}^*$	Ratio > 1.0	N/A N/A

*Not applicable for top stories, see AISC 341 E3.4a(a1))



Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	3422
Beam plastic section modulus	Z_b (in ³)	254
Beam plastic bending moment, $F_{yb}Z_b$	M_p (k-in)	12700
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	9535 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.875
Beam depth	d_b (in)	24.3
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	189.4
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	189.4
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	378.7

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Beam area	A_{gb} (in ²)	27.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	69.3
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	448.0
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	35
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.875
Beam depth	d_b (in)	24.3
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	497.1
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	1
Cover plate overhang dimension	C3 (in)	3.5
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	28.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	861.0
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	861.0
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.52 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	26.43
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	792.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	792.8
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.63 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	24.3
Beam flange thickness	t_{fb} (in)	0.875
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.55
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	2.13 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.125
Beam plastic section modulus	Z_b (in ³)	254
Beam flange thickness	t_{fb} (in)	0.875
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	1.094
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	51.2
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	202.8
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	13970
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	14497
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.96 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	138.5
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	69.3
Bolt diameter	d_{bolt} (in)	1
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.785
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	66.0
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$	$n_{b, req}$	3.9
Number of bolts per line (AISC 358 15.2(1))	n_b	6 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Bolt spacing (see schedule)	s (in)	3.25
Plate thickness, t_p (see T2 in drawings)	t_p (in)	0.875
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	2.312
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a] + 3in - 2e_d] / s + 1$	$n_{p, min}$	2.5
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	3

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	258.6
Beam flange thickness	t_{fb} (in)	0.875
Fuse and top plate thickness	$T2$ (in)	0.875
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	0.9
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	819.0
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	2.0
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	682.5
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o / 2$	l_{c2} (in)	0.8
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	51.2
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	733.7
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	395.8
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	395.8
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	296.9
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.87 OK

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451
Bolt group demand from wind per line, $M_u / (2(d_b + t_p))$	R_u (kip)	28.8
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	64
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	6
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	130.2
Total factored bolt slip resistance, $0.85 R_n$	ϕR_n (kips)	110.6
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.26 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

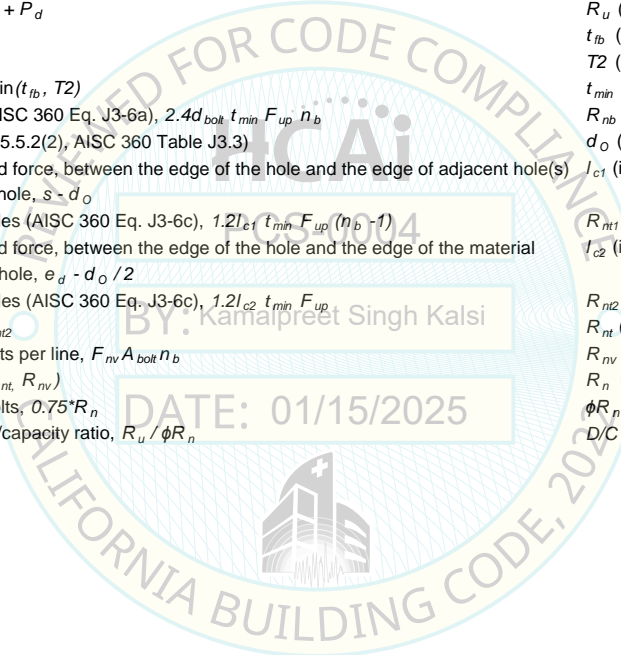
- Maximum probable force per bolt line (see step 2)
- Total drag force through fuse plate (all drag force transmitted through top plate)
- Drag force per line of bolts, $P_{d,total}/2$
- Bolt diameter
- Area of single bolt, $\pi \cdot d_{bolt}^2 / 4$
- Bolt shear strength, $\phi F_{nv} A_b$
- Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$
- Number of bolts per line on bottom flange (AISC 358 15.2(1))
- Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)
- Bolt spacing (see schedule)
- Plate thickness, t_p (see T2 in drawings)
- Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7),
 $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$
- Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6),
 $[2[F2_a] + 3in - 2e_d] / s + 1$
- Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)
- Number of bolts behind the alignment line on the outside lines on bottom flange (zone M)
(AISC 358 Eq. 15.6-8), $nb - np$

V_{fe} (kips)	189.4
$P_{d,fuse}$ (kips)	0.0
P_d (kips)	0.0
d_{bolt} (in)	1
A_{bolt} (in ²)	0.785
ϕr_n (kips)	66.0
$n_{b, req}$	2.9
$n_{b, Bot}$	5 OK
e_d (in)	1.375
s (in)	3.25
t_p (in)	0.875
$F2_a$ (in)	2.312
$n_{p,min}$	2.5
$n_{p, Bot}$	3 OK
$n_{m, Bot}$	2

Flange bolt limit states check

- Total force on bolts per line, $V_{fe} + P_d$
- Beam flange thickness
- Fuse and top plate thickness
- Governing material thickness, $\min(t_{fb}, T2)$
- Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$
- Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)
- Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s)
 I_{c1} is based on an oversized hole, $s - d_o$
- Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c1} t_{min} F_{up} (n_b - 1)$
- Clear distance, parallel to applied force, between the edge of the hole and the edge of the material
 I_{c2} is based on an oversized hole, $e_d - d_o / 2$
- Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c2} t_{min} F_{up}$
- Total tearout strength, $R_{nt1} + R_{nt2}$
- Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$
- Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$
- Factored shear capacity of all bolts, $0.75 \cdot R_n$
- Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$

R_u (kips)	189.4
t_{fb} (in)	0.875
$T2$ (in)	0.875
t_{min} (in)	0.9
R_{nb} (kips)	819.0
d_o (in)	1.25
I_{c1} (in)	2.0
R_{nt1} (kips)	546.0
I_{c2} (in)	0.8
R_{nt2} (kips)	51.2
R_{nt} (kips)	597.2
R_{nv} (kips)	329.9
R_n (kips)	329.9
ϕR_n (kips)	247.4
D/C	0.77 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.3
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	0
Cover plate overhang dimension	C3 (in)	3.5
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max}d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	2.833
Distance from alignment line to column	C1 (in)	6.875 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per weld, $P_{d,total}/2$	P_d (kips)	69.25
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	35
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.3
Plate thickness (see T2 on drawings)	t_p (in)	0.875
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	1
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	26.425
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	2.75
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	25.43
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.88
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	6
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.89
Vertical distance from center of rotation to geometric center	e_y (in)	1.47

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) \cdot L$ ΣR_L (k-in) 16056

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R \cdot L_R$ (k-in)
W1RT	17.13	7.62	54.42	130.5
W4TL	19.49	5.22	32.87	101.8
W1LB	16.08	7.26	43.11	116.6
W4BR	16.06	5.30	33.25	85.2
W1RC	10.11	5.24	9.82	53.0
W4TC	16.41	4.18	3.79	68.6
W1LC	11.86	5.33	8.36	63.3
W4BC	13.47	4.07	4.61	54.8
		7.62	54.42	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	7.6
Orientation of Resultant Force	θ_r (degrees)	54.4
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.51
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma R_L)$	r_{uv} (kip/in)	9.7
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c \cdot l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	9.7
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.30
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 \cdot 0.6 \cdot 70 \cdot (1 + 0.5 \cdot \sin^{1.5}(\theta_r)) \cdot (p_i \cdot (1.9 - 0.9p_i)))^{0.3} \cdot 0.707 \cdot D_{W1}/16$	ϕr_n (kips)	11.24
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.86 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	69.25
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	C3 (in)	3.5
External continuity plate dimension	C5 (in)	2.75
Weld 2 size	D_{W2} (1/16 in)	7
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.44
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	5.02
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	15.6
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3]) - 2l_{hb2} - l_{we2,eff})$	r_{un2} (kip/in)	10.8
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	19.0
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2 \text{ sides})$	ϕr_n (kip/in)	19.5
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	D/C	0.97 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1232.0
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	448.0
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c + 2[C3])t_p$	ϕR_n (kips)	753.4
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	8108.2
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.75 OK

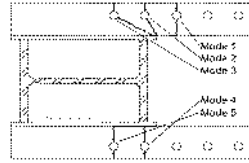
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	69.3
Column fillet dimension	k (in)	1.5
Column fillet dimension	$k1$ (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	93.2
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	7
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1210.9
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	566.4
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	1878.9
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1409.2
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.32 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	69.3
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	129.3
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continuity plate width	$C4$ (in)	4.75
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Net area of external continuity plate cross section, $t_p([C4]-d_{hole,std}+1/16)$	A_n (in ²)	3.117
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	202.6
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	152.0
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.85 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

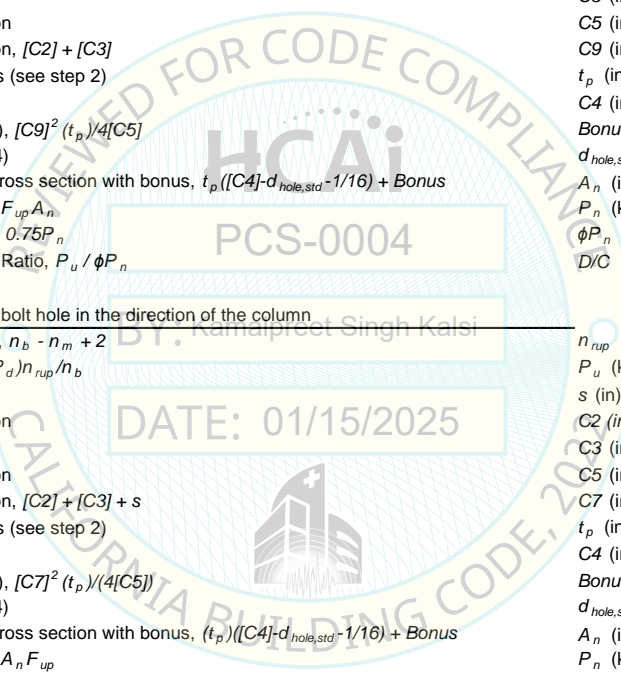
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	172.4
External continuity plate dimension	$C2$ (in)	-1.125
Cover plate overhang dimension	$C3$ (in)	3.5
External continuity plate dimension	$C5$ (in)	2.75
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.375
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continuity plate width	$C4$ (in)	4.75
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p)/4[C5]$	$Bonus$ (in)	0.449
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	3.566
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	231.8
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	173.8
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.99 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	215.5
Bolt spacing (see schedule)	s (in)	3.25
External continuity plate dimension	$C2$ (in)	-1.125
Cover plate overhang dimension	$C3$ (in)	3.5
External continuity plate dimension	$C5$ (in)	2.75
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.625
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continuity plate width	$C4$ (in)	4.75
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p)/(4[C5])$	$Bonus$ (in)	2.517
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Area of external continuity plate cross section with bonus, $(t_p)/([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in ²)	5.634
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	366.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	274.7
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.78 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	172.4
External continuity plate dimension	$C2$ (in)	-1.125
Cover plate overhang dimension	$C3$ (in)	3.5
External continuity plate width	$C4$ (in)	4.75
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.375
External continuity plate thickness (see step 2)	t_p (in)	0.875
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	4.156
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	2.078
Net tension area, $A_{gt} - ((t_p)^2(d_{hole,std}+1/16))$	A_{nt} (in ²)	3.117
Net shear area, A_{gv}	A_{nv} (in ²)	2.078
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	265.0
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	198.7
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.87 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 5 rupture demand, $(V_{fe} + P_d) \cdot n_{rup} / n_b$	R_u (kips)	215.5
Bolt spacing (see schedule)	s (in)	3.25
External continuity plate dimension	C2 (in)	-1.125
Cover plate overhang dimension	C3 (in)	3.5
External continuity plate width	C4 (in)	4.75
External continuity plate dimension, $[C2] + [C3] + s$	C7 (in)	5.625
External continuity plate thickness (see step 2)	t_p (in)	0.875
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.125
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	4.156
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	4.922
Net tension area, $A_{gt} - (t_p (d_{hole} + 1/16))$	A_{nt} (in ²)	3.117
Net shear area, A_{gv}	A_{nv} (in ²)	4.922
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	350.3
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	262.7
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.82 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	7.0
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	9535
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	63.4
Beam depth	d_b (in)	24.3
Beam web thickness	t_{wb} (in)	0.515
Beam web area, $d_b t_{wb}$	A_w (in)	12.5145
Beam web width-to-thickness ratio, h / t_w	l_{web}	41.9
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_wC_v$	ϕV_n (kips)	375.4
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.17 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	138.5
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	69.3
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	517.2
Beam flange thickness	t_{fb} (in)	0.875
Beam flange width	b_{fb} (in)	9.07
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.125
Number of holes along each bolt line (see step 5)	n_b	6
Length of connection on beam	B4 (in)	17.625
Distance from beam centerline to bolt line	P2 (in)	2.75
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	3.124
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	30.844
Net tension area, $A_{gt} - (t_{fb} (d_{hole, std} + 1/16))$	A_{nt} (in ²)	2.085
Net shear area, $A_{gv} - (t_{fb} (d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	19.414
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	892.7
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	669.5
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.77 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

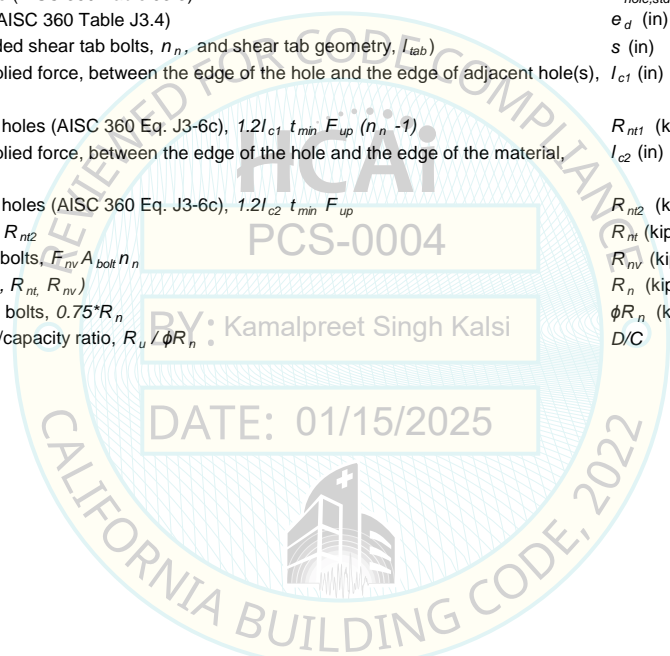
The required strength of shear tab bolt group (see step 10)	R_u (kips)	63.4
Bolt diameter (see step 4)	d_{bolt} (in)	1
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.785
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	66.0
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	49.5
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	1
Provided number of shear tab bolts	n_n	4 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.515
Shear tab thickness	$T3$ (in)	0.5
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	312.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.250
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.500
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.4
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	511.9
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.7
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	26.8
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	538.7
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	263.9
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	263.9
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	197.9
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.32 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	64
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	32.5
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	130.2
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	63.4
Distance from alignment line to column	$C1$ (in)	6.875
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	499.8

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Slotted bolt hole width (standard slot width)	w_b (in)	1.125
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.25
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.375
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	7.13
Effective net area, $A_e = A_{nv}$	A_e (in ²)	7.13
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	208.41
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	347.34
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.15 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	63.37
Required axial strength of the shear tab (see above)	P_u (kips)	130.18
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	499.83
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{ny} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nt} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nt})^2 + (V_u/\phi R_{ny})^4$	D/C	0.34 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.55
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	63.4
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	2.8
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.375
Distance from line of bolts to shear tab weld	$C1$ (in)	6.875
Size of weld in 1/16 ^{ths} of an inch	D_{w3}	3
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	499.8
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u) / l_{w3}^2 - 2F_b n_{vb} / l_{w3}$ except when bridge plates present, $F_b n_{vb} / l_{w3}$	r_{un} (k/in)	5.8
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	6.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	8.4
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.77 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.375
Beam depth	d_b (in)	24.3
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.7165
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.75 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.75

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	69.3
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	258.6

Yielding check

Top plate thickness (see step 2)	t_p (in)	0.875
Top plate dimension (see schedule)	$P6$ (in)	20.25
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	17.719
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	531.6
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	531.6
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.49 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.3125
Number of short slotted bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	10.828
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	422.3
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	316.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.82 OK

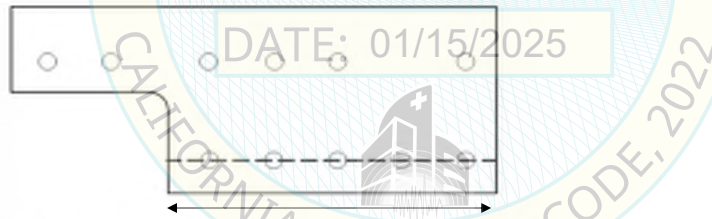


Fig. C-15.8. Shear rupture of net section on top plate.

Note: For the alternate top plate, this check is conservative and does not need to be checked for the top plate.

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	69.3
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	258.6

Yielding check (LS 1)

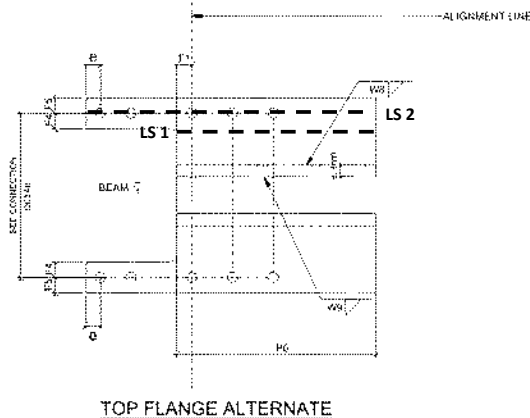
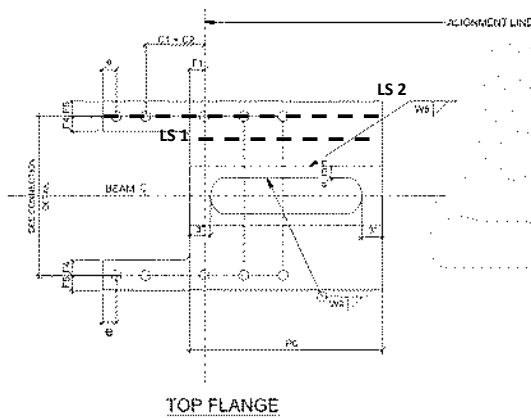
Top plate thickness (see step 2)	t_p (in)	0.875
Top plate dimension (see schedule)	$P6$ (in)	20.25
Fuse plate dimension	$F1$ (in)	2.00
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	17.719
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	531.6
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	531.6
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.49 OK

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Number of oversized bolts in Zone M (see step 5)	n_m	3
Number of oversized bolts on top plate (see step 5)	n_b	6
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.25
Bolt spacing (see schedule)	s (in)	3.25
Fuse plate dimension	$F1$ (in)	2.00
Top plate dimension, $e_d + s*(n_m-1) + C1 + C2 + -F1 + P6$	$Dim 1$ (in)	31.75
Gross shear area of top plate, $(Dim 2)(t_p)$	A_g (in ²)	27.781
Number of oversized bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g - (t_p d_o n_b)$	A_n (in ²)	21.219
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	827.5
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	620.6
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.42 OK

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u (kips)	258.6
Beam flange width	b_{fb} (in)	9.07
Weld 8 size	D_{W8} (1/16 in)	4
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.25
Weld 8 length, $P6 - 2l_{hb8}$	l_{w8} (in)	19.75
Weld 9 size	D_{W9} (1/16 in)	4
Weld 9 length, $l_{w8} - 2*3"$	l_{w9} (in)	13.75
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u / (l_{w8} + l_{w9})$	r_{uv} (kip/in)	7.7
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16 + D_{W9}/16)(1")$	ϕr_n (kip/in)	11.1
Alternative top plate-to-beam flange weld tearing check, $r_{uv} / \phi r_n$	D/C	0.69 OK



Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	69.3
Total number of bolts (see step 5)	n_b	6
Number of bolts in the M region (see step 5)	n_m	3
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	129.3

Yielding check

Top plate dimension	$P4$ (in)	1.875
Top plate dimension	$P5$ (in)	2.5
Top plate thickness (see step 2)	t_p (in)	0.875
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	3.828
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	191.4
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	172.3
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.75 OK

Rupture check

Top plate dimension	$P4$ (in)	1.875
Top plate dimension	$P5$ (in)	2.5
Top plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	2.680
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_u A_e$	P_n (kips)	174.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	130.6
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99 OK

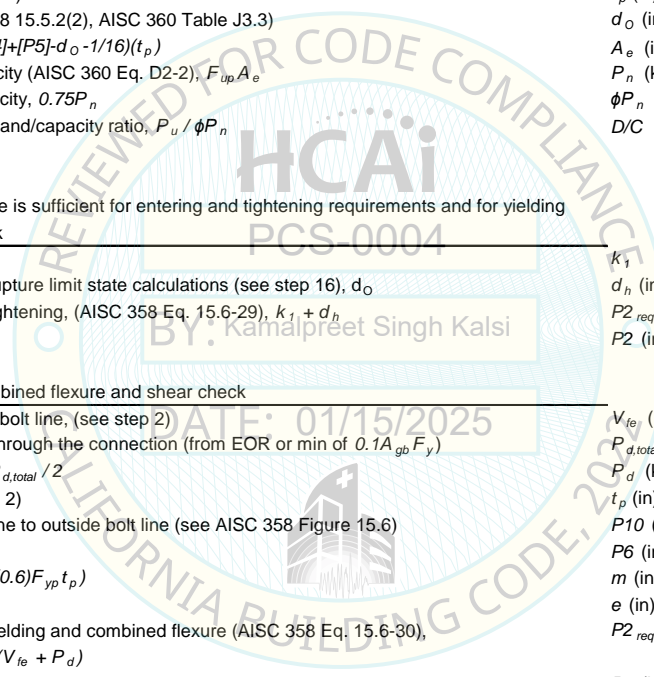
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.4
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.25
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.6875
Provided P2 dimension	$P2$ (in)	2.75 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	69.3
Top plate thickness (see step 2)	t_p (in)	0.875
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	9.95
Inside length of the top plate	$P6$ (in)	20.25
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	10.947
Dimension, $([P6] - m) / 2$	e (in)	4.651
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-1.1
Provided P2 dimension	$P2$ (in)	2.75 OK



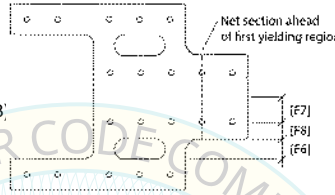
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.375 OK
Top plate thickness (see step 2)	t_p (in)	0.875
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.7 OK

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

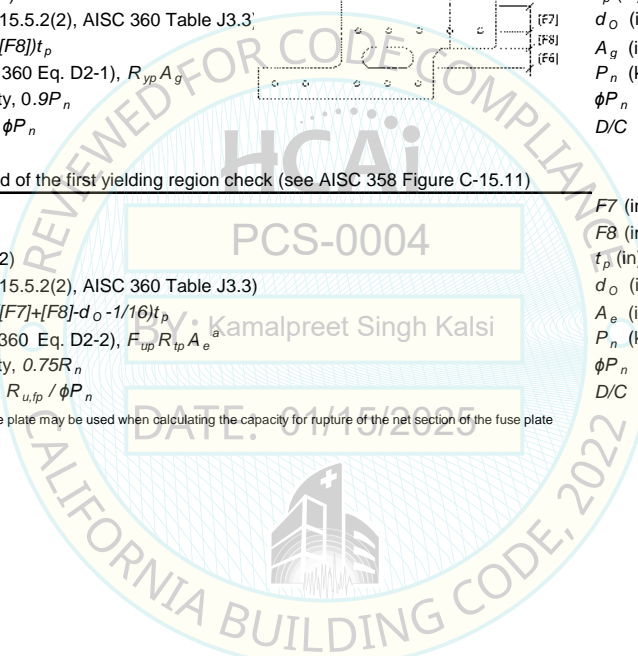
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	5
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	2
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	151.5
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	9.975
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	498.8
Factored tensile capacity, $0.9P_n$	ϕP_n (kips)	448.9
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.34 OK



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.250
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	7.678
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$	P_n (kips)	598.9
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	449.2
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.34 OK

*The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	189.4
Fuse plate thickness(see step 2)	t_p (in)	0.875
Width of fuse plate cut-out	$F6$ (in)	2.375
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6])/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.020
Provided yielding region depth	$F2$ (in)	2.000 OK
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	105.0
Ratio	V_{fe} / V_y	1.8
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	1.19 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	5
Number of bolts in the M region	$n_{m, Bot}$	2
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	151.5
Yielding check		
Fuse plate dimension	$F4$ (in)	1.875
Fuse plate dimension	$F5$ (in)	1.375
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	5.6875
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	284.38
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	255.94
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	1.875
Fuse plate dimension	$F5$ (in)	1.375
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	3.390625
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	264.5
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	198.4
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.76 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	138.5
Drag force per line of bolts, $P_d / 2$	P_d (kips)	69.25
External continuity plate dimension	C5 (in)	2.75
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	28.2
Cover plate overhang	C3 (in)	3.5
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1)$	ϕR_n (kips)	70.3
Tension failure demand/capacity ratio, $P_u/\phi R_n$	D/C	0.40 OK

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	28.2
Weld 5 size	D_{W5} (1/16 in)	5
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W5}/16([C3]-1)(2 \text{ sides})$	ϕR_n (kips)	52.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.54 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.3
Shear force from gravity	$V_{gravity}$ (kips)	7.0
Maximum probable moment at fuse location (see Step 2)	M_{pr} (K-in)	9535
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	63.4
Distance from alignment line to column	C1 (in)	6.875
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	17.9
Weld 7 size	D_{W7} (1/16 in)	3
Cover plate overhang	C3 (in)	3.5
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_{W7}/16[C3](2 \text{ sides})$	ϕR_n (kips)	29.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.61 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	63.4
Distance from alignment line to column	C1 (in)	6.875
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	17.9
Weld 6 size	D_{W6} (1/16 in)	4
Cover plate overhang	C3 (in)	3.5
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	7
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W6}/16(l_{eff})$	ϕR_n (kips)	58.5
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.31 OK

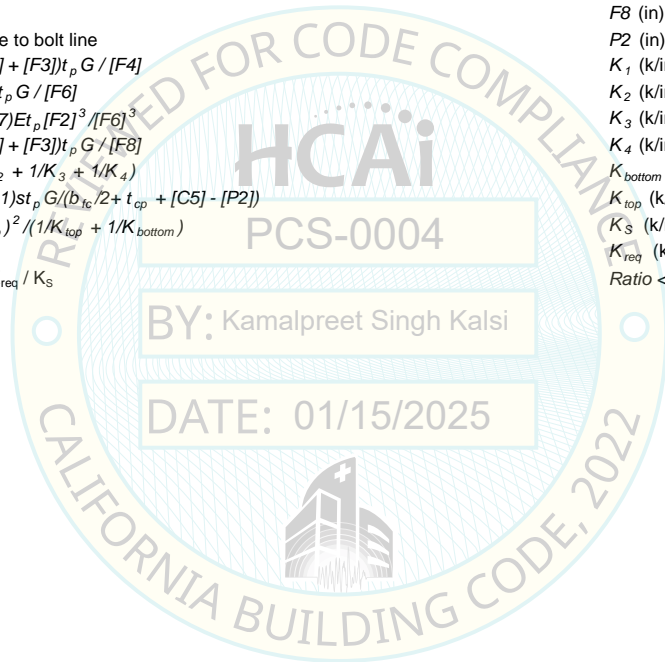
Shear plate shear failure check (when $W1 > 3/4"$ without shear plates, shear plates are used)

Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Weld 4 size	D_{W4} (1/16 in)	6
Weld 4 effective length, $d_c - 2 t_{fc}$	l_{eff} (in)	19.78
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16(l_{eff})$	R_u (kips)	165.2
Shear plate thickness, $t_{sp} \geq 1.1W4 \text{ or } 3/8"$	t_{sp} (in)	0.500
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c - 2t_{fc})$	ϕR_n (kips)	296.7
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6 OK

Connection stiffness check

The purpose of this check is to ensure that the connection has enough stiffness to be considered Fully Restrained (FR) when M_{pr} is less than M_p (see AISC 358-22 15.6 Step 2). Moment frame connections are considered FR for design purposes if the connection stiffness is large relative to the beam flexural stiffness (EI/L). A threshold of $18EI/L$ is used in Chapter 13 of AISC 358 based on experimental testing. This procedure follows the method in the commentary to AISC 358-22 15.6 Step 2.

Beam moment of inertia	I_{xb} (in ³)	2700
Column flange width	b_{fc} (in)	12.4
Beam depth	d_b (in)	24.3
Bay width	B (in)	360.0
Number of bolts in zone M	M	3
Number of bolts in zone P	P	3
Bolt spacing	s (in)	3.25
Cover plate thickness	t_{cp} (in)	1
Fuse plate thickness	t_p (in)	0.875
External continuity plate dimension	C5 (in)	2.75
Fuse plate dimension	F2 (in)	2
Fuse plate dimension	F3 (in)	5.875
Fuse plate dimension	F4 (in)	1.9
Fuse plate dimension	F6 (in)	2.375
Fuse plate dimension	F8 (in)	2.95
Distance from beam centerline to bolt line	P2 (in)	2.75
Fuse region 1 stiffness, $(2[F2] + [F3])t_p G / [F4]$	K_1 (k/in)	5.16E+04
Fuse region 2 stiffness, $2[F2]t_p G / [F6]$	K_2 (k/in)	1.65E+04
Fuse region 3 stiffness, $2(0.37)Et_p [F2]^3 / [F6]^3$	K_3 (k/in)	1.12E+04
Fuse region 4 stiffness, $(2[F2] + [F3])t_p G / [F8]$	K_4 (k/in)	3.28E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	1.00E+04
Top plate stiffness, $(2P + M - 1)st_p G / (b_{fc}/2 + t_{cp} + [C5] - [P2])$	K_{top} (k/in)	3.54E+04
Connection stiffness, $(d_b + t_p)^2 / (1/K_{top} + 1/K_{bottom})$	K_S (k/in)	4.95E+06
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	3.92E+06
Connection stiffness check, K_{req} / K_S	Ratio < 1	0.79 OK



Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

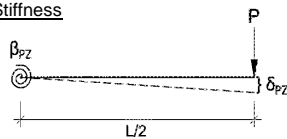
Show that effective panel zone stiffness $\geq 10 \times$ beam flexural stiffness

Effective Panel Zone Stiffness

$$K_{PZ} = P/\delta_{PZ}$$

$$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$$

$$K_{PZ} = \beta_{PZ}/(L/2)^2$$

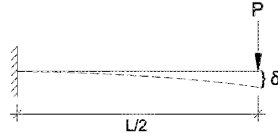


Beam Stiffness

$$K_B = P/\delta_B$$

$$\delta_B = P(L/2)^3/3EI$$

$$K_B = 3EI/(L/2)^3$$

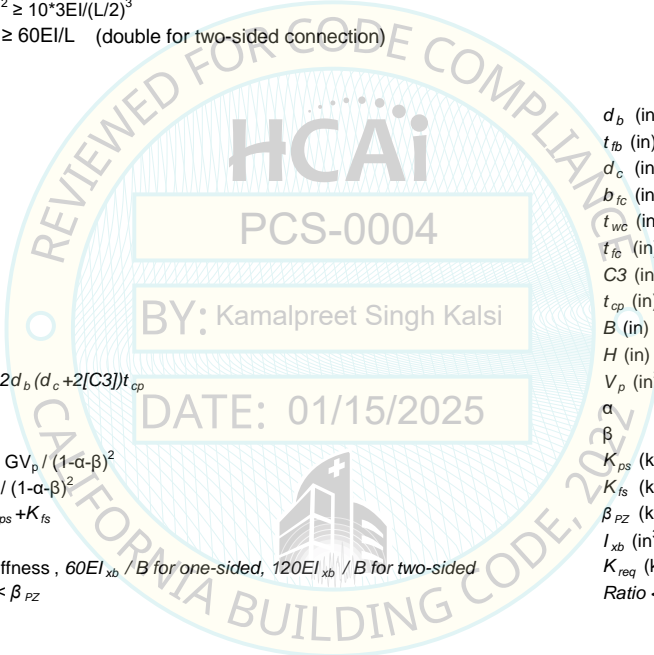


$$K_{PZ} \geq 10K_B$$

$$\beta_{PZ}/(L/2)^2 \geq 10 \times 3EI/(L/2)^3$$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.3
Beam flange thickness	t_{fb} (in)	0.875
Column depth	d_c (in)	21.7
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.6
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang	C3 (in)	3.5
Cover plate thickness	t_{cp} (in)	1
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])t_{cp}$	V_p (in ³)	1711.2
Column ratio, $(d_c - t_{fc}) / B$	α	0.058
Beam ratio, $(d_b - t_{fb}) / H$	β	0.108
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	2.76E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.77E+07
Beam moment of inertia	I_{xb} (in ³)	2700
Required panel zone spring stiffness, $60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided	K_{req} (k-in)	2.61E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	Ratio < 1	0.94 OK



DuraFuse Frames® Calculations

HCAI CALCULATIONS

Connection ID DF251
 W21X122 Column and W24X76 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200

Beams

Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1

Columns

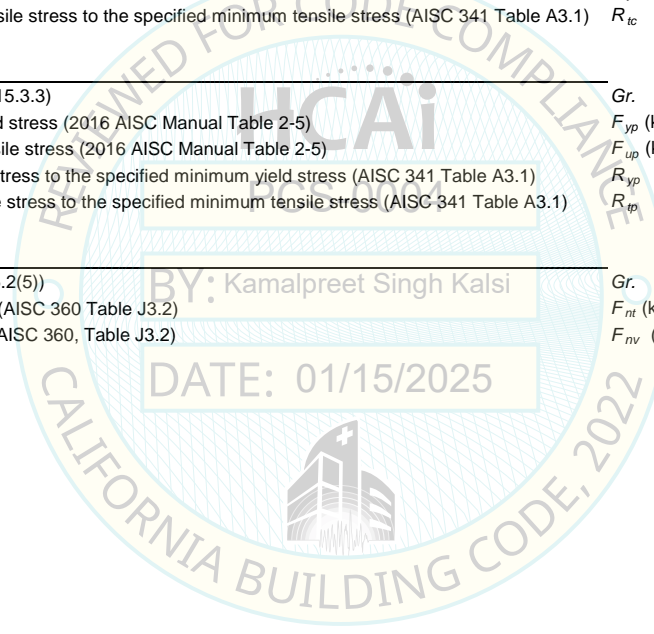
Column material grade	<i>Gr.</i>	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.1

Plates

Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2

Bolts

Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

		W24X76
Beam size		
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	23.9 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	76 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	49 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_y)^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	6.61 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))	<i>See bracing calculations</i>	

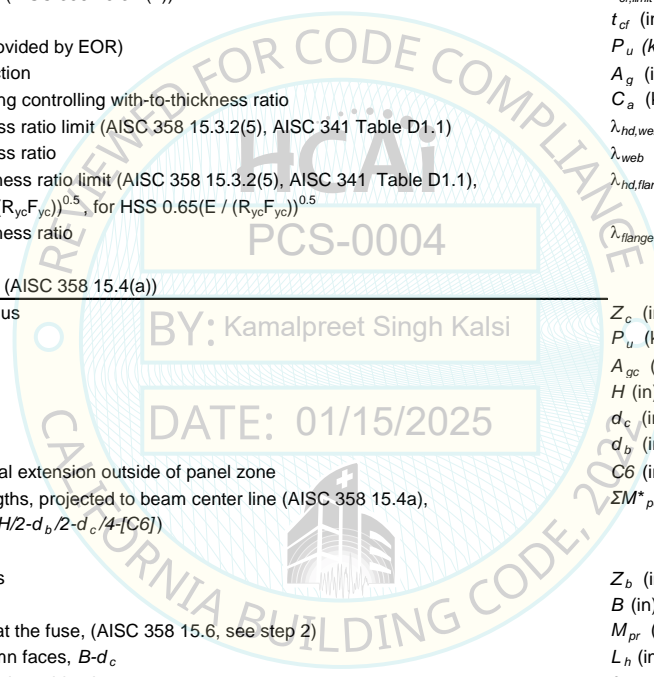
Column limitation checks

		W21X122
Column size		
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	21.7
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	122 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.96 OK
Axial force on the column (provided by EOR)	P_u (kips)	53.5
Gross area of the column section	A_g (in ²)	35.9
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.0
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	57.2
Column web width-to-thickness ratio	λ_{web}	31.3 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35
Column flange width-to-thickness ratio	λ_{flange}	6.45 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	307
Column design axial load	P_u (kips)	53.5
Column area	A_{gc} (in ²)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	23.9
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	3.67E+04
Beam plastic section modulus	Z_b (in ³)	200
Minimum bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	10000
Clear distance between column faces, $B - d_c$	L_h (in)	338.3
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	7.0
Beam shear (AISC 358 15.4a), $2M_{pr} / L_h + V_g$	V_b (kips)	66.1
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	717
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	2.14E+04
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	<i>Ratio > 1.0</i>	1.71 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1))



Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	3422
Beam plastic section modulus	Z_b (in ³)	200
Beam plastic bending moment, $F_y Z_b$	M_p (k-in)	10000
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	10000 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1
Beam depth	d_b (in)	23.9
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	200.8
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	6999.6
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	140.6
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	341.4

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	341.4
Beam area	A_{gb} (in ²)	22.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	56.0
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	397.4
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	35
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1
Beam depth	d_b (in)	23.9
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	441.3
<u>Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)</u>		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhang dimension	C3 (in)	3.5
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	28.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	645.8
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	645.8
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.62 OK
<u>Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)</u>		
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	29.90
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	672.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	672.8
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.66 OK
<u>Cover plate thickness check (AISC 341 E3.6e.2)</u>		
Beam depth	d_b (in)	23.9
Beam flange thickness	t_{fb} (in)	0.68
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.54
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.59 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.125
Beam plastic section modulus	Z_b (in ³)	200
Beam flange thickness	t_{fb} (in)	0.68
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	0.850
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	39.5
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	160.5
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	11000
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	11478
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.96 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	112.0
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	56.0
Bolt diameter	d_{bolt} (in)	1
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.785
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	66.0
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$	$n_{b, req}$	3.9
Number of bolts per line (AISC 358 15.2(1))	n_b	6 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Bolt spacing (see schedule)	s (in)	3.25
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	2.145
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a] + 3in - 2e_d] / s + 1$	$n_{p, min}$	2.4
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	3

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	256.8
Beam flange thickness	t_{fb} (in)	0.68
Fuse and top plate thickness	$T2$ (in)	1
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	0.7
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	636.5
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	2.0
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	530.4
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o / 2$	l_{c2} (in)	0.8
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	39.8
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	570.2
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	395.8
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	395.8
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	296.9
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.87 OK

Bolt slip for wind load check

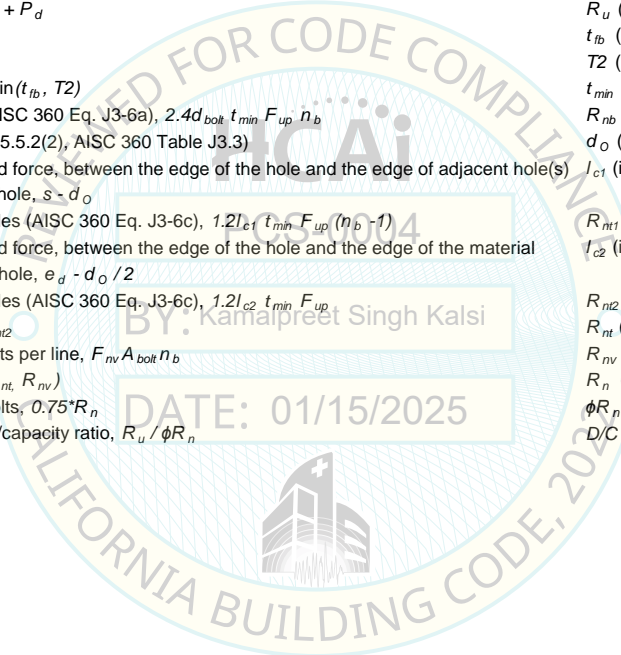
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451
Bolt group demand from wind per line, $M_u / (2(d_b + t_p))$	R_u (kip)	29.1
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	64
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	6
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	130.2
Total factored bolt slip resistance, $0.85 R_n$	ϕR_n (kips)	110.6
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.26 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	200.8
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0
Bolt diameter	d_{bolt} (in)	1
Area of single bolt, $\pi \cdot d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.785
Bolt shear strength, $\phi F_{nv} A_b$	ϕR_n (kips)	66.0
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi R_n$	$n_{b, req}$	3.0
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b, Bot}$	5 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Bolt spacing (see schedule)	s (in)	3.25
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	2.145
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a] + 3in - 2e_d] / s + 1$	$n_{p, min}$	2.4
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p, Bot}$	3 OK
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), $nb - np$	$n_{m, Bot}$	2

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	200.8
Beam flange thickness	t_{fb} (in)	0.68
Fuse and top plate thickness	$T2$ (in)	1
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	0.7
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	636.5
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) I_{c1} is based on an oversized hole, $s - d_o$	I_{c1} (in)	2.0
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	424.3
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material I_{c2} is based on an oversized hole, $e_d - d_o / 2$	I_{c2} (in)	0.8
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2I_{c2} t_{min} F_{up}$	R_{nt2} (kips)	39.8
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	464.1
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	329.9
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	329.9
Factored shear capacity of all bolts, $0.75 \cdot R_n$	ϕR_n (kips)	247.4
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.81 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	23.9
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	0
Cover plate overhang dimension	C3 (in)	3.5
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max}d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	2.809
Distance from alignment line to column	C1 (in)	6.375 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	341.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	56
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	35
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	23.9
Plate thickness (see T2 on drawings)	t_p (in)	1
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	29.9
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	2.75
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.4375
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.03
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.75
Weld 1 size	D_{W1} (1/16 in)	5
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	5
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.97
Vertical distance from center of rotation to geometric center	e_y (in)	1.31

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d) \cdot L$ ΣR_L (k-in) 14742

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R \cdot L_R$ (k-in)
W1RT	18.41	7.37	57.55	135.8
W4TL	19.24	4.42	33.66	85.1
W1LB	17.50	7.08	47.51	123.9
W4BR	15.98	4.40	33.08	70.3
W1RC	10.01	4.83	9.23	48.4
W4TC	16.06	3.42	4.18	54.9
W1LC	11.93	4.94	7.73	58.9
W4BC	13.44	3.34	4.99	44.8
		7.37	57.55	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	7.4
Orientation of Resultant Force	θ_r (degrees)	57.6
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.58
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma R_L)$	r_{uv} (kip/in)	7.9
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c \cdot l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	7.9
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.28
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75 \cdot 0.6 \cdot 70 \cdot (1 + 0.5 \cdot \sin^{1.5}(\theta_r)) \cdot (p_i \cdot (1.9 - 0.9p_i)))^{0.3} \cdot 0.707 \cdot D_{W1}/16$	ϕr_n (kips)	9.53
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.83 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	341.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	C3 (in)	3.5
External continuity plate dimension	C5 (in)	2.75
Weld 2 size	D_{W2} (1/16 in)	7
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.44
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.77
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	13.8
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3]) - 2l_{hb2} - l_{we2,eff})$	r_{un2} (kip/in)	9.9
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	17.0
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2 \text{ sides})$	ϕR_n (kip/in)	19.5
External continuity plate-to-cover plate weld tearing check, $r_u / \phi R_n$	D/C	0.87 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	341.4
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1092.7
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	397.4
External continuity plate thickness (see step 2)	t_p (in)	1.000
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c + 2[C3])t_p$	ϕR_n (kips)	861.0
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	9266.5
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.58 OK

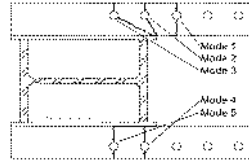
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	341.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Column fillet dimension	k (in)	1.5
Column fillet dimension	$k1$ (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	93.2
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	7
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1210.9
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	566.4
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	1878.9
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1409.2
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.28 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	128.4
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	$C4$ (in)	4.5
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Net area of external continuity plate cross section, $t_p([C4] - (d_{hole,std} + 1/16))$	A_n (in ²)	3.313
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	215.3
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	161.5
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.80 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

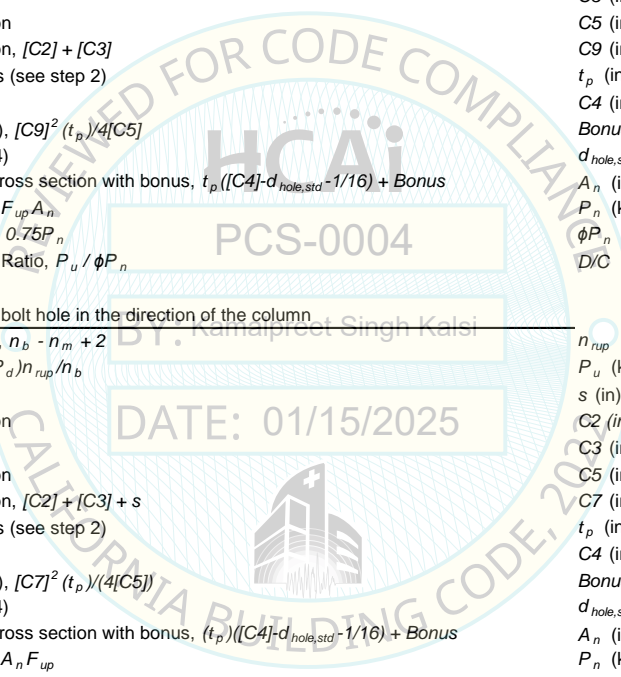
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	171.2
External continuity plate dimension	$C2$ (in)	-1.625
Cover plate overhang dimension	$C3$ (in)	3.5
External continuity plate dimension	$C5$ (in)	2.75
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	1.875
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	$C4$ (in)	4.5
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p) / (4[C5])$	$Bonus$ (in)	0.320
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Area of external continuity plate cross section with bonus, $t_p([C4] - d_{hole,std} - 1/16) + Bonus$	A_n (in ²)	3.632
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	236.1
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	177.1
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.97 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	214.0
Bolt spacing (see schedule)	s (in)	3.25
External continuity plate dimension	$C2$ (in)	-1.625
Cover plate overhang dimension	$C3$ (in)	3.5
External continuity plate dimension	$C5$ (in)	2.75
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.125
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	$C4$ (in)	4.5
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p) / (4[C5])$	$Bonus$ (in)	2.388
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Area of external continuity plate cross section with bonus, $(t_p)/([C4] - d_{hole,std} - 1/16) + Bonus$	A_n (in ²)	5.700
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	370.5
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	277.9
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.77 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	171.2
External continuity plate dimension	$C2$ (in)	-1.625
Cover plate overhang dimension	$C3$ (in)	3.5
External continuity plate width	$C4$ (in)	4.5
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	1.875
External continuity plate thickness (see step 2)	t_p (in)	1
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Gross tension area, $t_p[C4]$	A_{gt} (in ²)	4.500
Gross shear area, $t_p[C9]$	A_{gv} (in ²)	1.875
Net tension area, $A_{gt} - ((t_p)^2(d_{hole,std} + 1/16))$	A_{nt} (in ²)	3.313
Net shear area, A_{gv}	A_{nv} (in ²)	1.875
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	271.6
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	203.7
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.84 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 5 rupture demand, $(V_{fe} + P_d) \cdot n_{rup} / n_b$	R_u (kips)	214.0
Bolt spacing (see schedule)	s (in)	3.25
External continuity plate dimension	C2 (in)	-1.625
Cover plate overhang dimension	C3 (in)	3.5
External continuity plate width	C4 (in)	4.5
External continuity plate dimension, $[C2] + [C3] + s$	C7 (in)	5.125
External continuity plate thickness (see step 2)	t_p (in)	1
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	1.125
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	4.500
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	5.125
Net tension area, $A_{gt} - (t_p (d_{hole} + 1/16))$	A_{nt} (in ²)	3.313
Net shear area, A_{gv}	A_{nv} (in ²)	5.125
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	369.1
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	276.8
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.77 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	7.0
Maximum probable moment at fuse location (see Step 2)	M_{pr} (K-in)	10000
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	66.1
Beam depth	d_b (in)	23.9
Beam web thickness	t_{wb} (in)	0.44
Beam web area, $d_b t_{wb}$	A_w (in)	10.516
Beam web width-to-thickness ratio, h / t_w	l_{web}	49
Beam web shear buckling coefficient (AISC 360 G2.1)	K_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_wC_v$	ϕV_n (kips)	315.5
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.21 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	112.0
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	56.0
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	513.6
Beam flange thickness	t_{fb} (in)	0.68
Beam flange width	b_{fb} (in)	8.99
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	1.125
Number of holes along each bolt line (see step 5)	n_b	6
Length of connection on beam	B4 (in)	17.625
Distance from beam centerline to bolt line	P2 (in)	2.75
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	2.373
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	23.970
Net tension area, $A_{gt} - (t_{fb} (d_{hole, std} + 1/16))$	A_{nt} (in ²)	1.566
Net shear area, $A_{gv} - (t_{fb} (d_{hole, std} + 1/16))(2n_b - 1)$	A_{nv} (in ²)	15.088
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	690.2
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	517.6
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.99 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

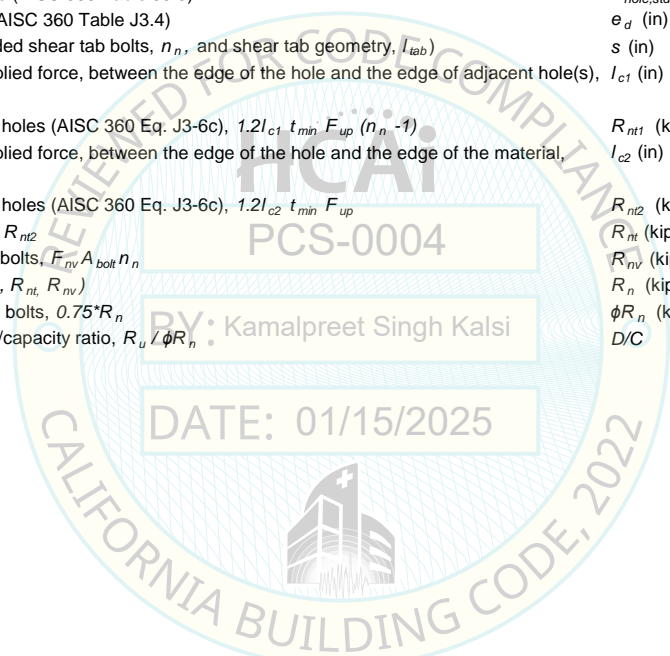
The required strength of shear tab bolt group (see step 10)	R_u (kips)	66.1
Bolt diameter (see step 4)	d_{bolt} (in)	1
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.785
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	66.0
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	49.5
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	1
Provided number of shear tab bolts	n_n	4 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.44
Shear tab thickness	$T3$ (in)	0.5
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.4
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	274.6
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.250
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	5.500
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.4
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	450.5
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.7
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	23.6
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	474.0
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	263.9
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	263.9
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	197.9
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.33 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	64
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	32.5
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	130.2
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	66.1
Distance from alignment line to column	$C1$ (in)	6.375
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	421.5

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Slotted bolt hole width (standard slot width)	w_b (in)	1.125
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.25
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.375
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in ²)	7.13
Effective net area, $A_e = A_{nv}$	A_e (in ²)	7.13
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	208.41
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{tn} (kips)	347.34
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.15 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	66.12
Required axial strength of the shear tab (see above)	P_u (kips)	130.18
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	421.51
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{tn} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.30 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	21.9
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	66.1
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	3.0
Number of slotted bolt holes in shear tab	n_{vb}	4
Slotted bolt hole spacing	s_{slot} (in)	5.375
Distance from line of bolts to shear tab weld	$C1$ (in)	6.375
Size of weld in 1/16 ^{ths} of an inch	D_{w3}	3
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u[(C1)+d_b/24]$ for roof sloped condition	M_u (k-in)	421.5
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u) / l_{w3}^2 - 2F_b n_{vb} / l_{w3}$ except when bridge plates present, $F_b n_{vb} / l_{w3}$	r_{un} (k/in)	5.9
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	6.7
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	8.4
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.80 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.375
Beam depth	d_b (in)	23.9
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.7045
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.75 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.75

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	256.8

Yielding check

Top plate thickness (see step 2)	t_p (in)	1
Top plate dimension (see schedule)	$P6$ (in)	19.5
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	19.500
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	585.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	585.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.44 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.3125
Number of short slotted bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	11.625
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	453.4
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	340.0
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.76 OK



Fig. C-15.8. Shear rupture of net section on top plate.

Note: For the alternate top plate, this check is conservative and does not need to be checked for the top plate.

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	256.8

Yielding check (LS 1)

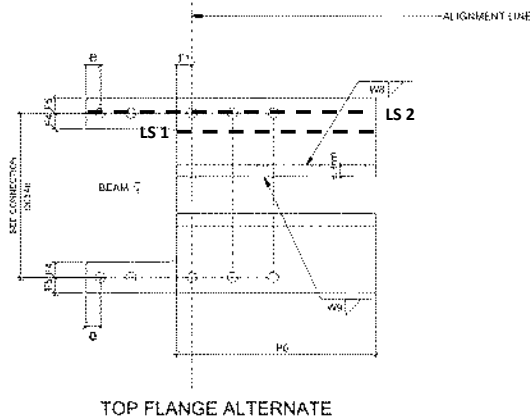
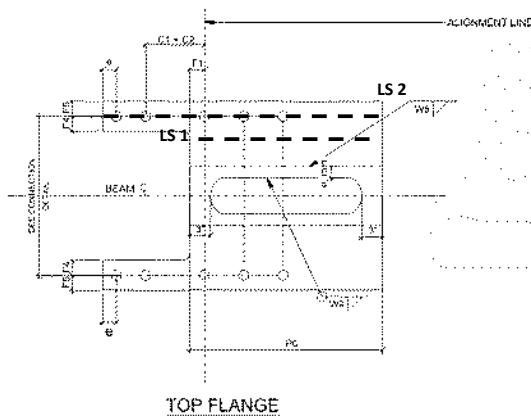
Top plate thickness (see step 2)	t_p (in)	1
Top plate dimension (see schedule)	$P6$ (in)	19.5
Fuse plate dimension	$F1$ (in)	1.63
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	19.500
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	585.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	585.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.44 OK

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Number of oversized bolts in Zone M (see step 5)	n_m	3
Number of oversized bolts on top plate (see step 5)	n_b	6
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.25
Bolt spacing (see schedule)	s (in)	3.25
Fuse plate dimension	$F1$ (in)	1.63
Top plate dimension, $e_d + s*(n_m-1) + C1 + C2 + -F1 + P6$	$Dim 1$ (in)	30.375
Gross shear area of top plate, $(Dim 2)(t_p)$	A_g (in ²)	30.375
Number of oversized bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g - (t_p d_o n_b)$	A_n (in ²)	22.875
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	892.1
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	669.1
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.38 OK

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u (kips)	256.8
Beam flange width	b_{fb} (in)	8.99
Weld 8 size	D_{W8} (1/16 in)	4
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.25
Weld 8 length, $P6 - 2l_{hb8}$	l_{w8} (in)	19.00
Weld 9 size	D_{W9} (1/16 in)	4
Weld 9 length, $l_{w8} - 2*3"$	l_{w9} (in)	13.00
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u / (l_{w8} + l_{w9})$	r_{uv} (kip/in)	8.0
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16 + D_{W9}/16)(1")$	ϕr_n (kip/in)	11.1
Alternative top plate-to-beam flange weld tearing check, $r_{uv} / \phi r_n$	D/C	0.72 OK



Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Total number of bolts (see step 5)	n_b	6
Number of bolts in the M region (see step 5)	n_m	3
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	128.4

Yielding check

Top plate dimension	$P4$ (in)	1.5
Top plate dimension	$P5$ (in)	2.5
Top plate thickness (see step 2)	t_p (in)	1
Gross area, $([P4]+[P5])*(t_p)$	A_g (in ²)	4.000
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	200.0
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	180.0
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.71 OK

Rupture check

Top plate dimension	$P4$ (in)	1.5
Top plate dimension	$P5$ (in)	2.5
Top plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Effective area of rupture, $([P4]+[P5]-d_o-1/16)(t_p)$	A_e (in ²)	2.688
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	174.7
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	131.0
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 OK

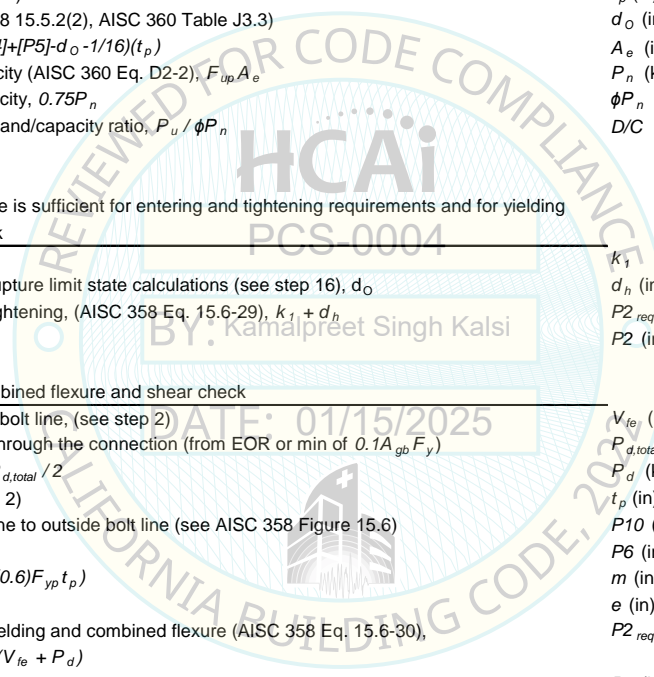
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	1.4
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	1.25
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.6875
Provided P2 dimension	$P2$ (in)	2.75 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Top plate thickness (see step 2)	t_p (in)	1
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	9.7
Inside length of the top plate	$P6$ (in)	19.50
Dimension, $(V_{fe} + P_d) / ((0.9)(0.6)F_{yp}t_p)$	m (in)	9.511
Dimension, $([P6] - m) / 2$	e (in)	4.994
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10] - (0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-3.0
Provided P2 dimension	$P2$ (in)	2.75 OK



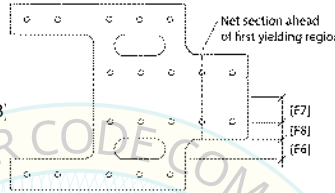
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.125 OK
Top plate thickness (see step 2)	t_p (in)	1
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.1 OK

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

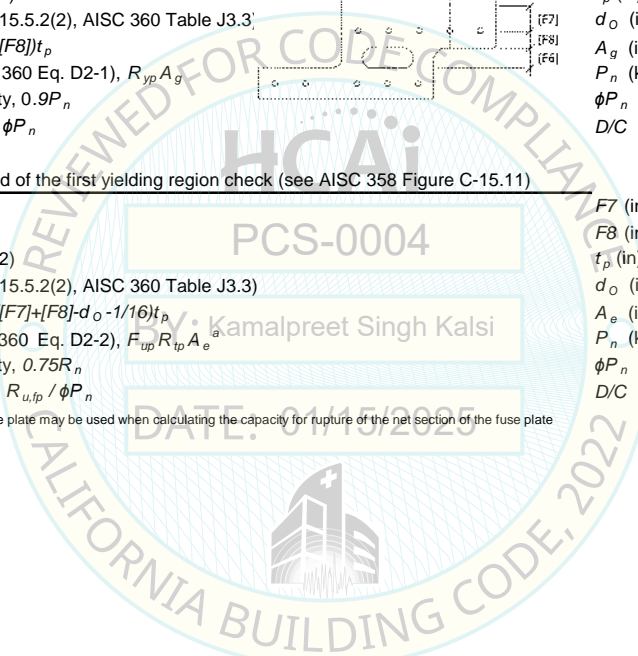
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	5
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	2
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	160.6
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	3.325
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	12.150
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	607.5
Factored tensile capacity, $0.9P_n$	ϕP_n (kips)	546.8
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.29 OK



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	3.325
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.250
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	9.525
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$	P_n (kips)	743.0
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	557.2
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.29 OK

*The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	200.8
Fuse plate thickness(see step 2)	t_p (in)	1
Width of fuse plate cut-out	$F6$ (in)	2.125
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B/[F6])/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	1.790
Provided yielding region depth	$F2$ (in)	1.750 OK
Fuse yield force (one side), $2(0.6F_{yp})/[F2]t_p$	V_y (kips)	105.0
Ratio	V_{fe} / V_y	1.9
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	1.21 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	5
Number of bolts in the M region	$n_{m, Bot}$	2
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fn}$ (kips)	160.6
Yielding check		
Fuse plate dimension	$F4$ (in)	1.5
Fuse plate dimension	$F5$ (in)	1.375
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	5.75
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	287.50
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	258.75
Yielding demand/capacity ratio, $R_{u,fn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	1.5
Fuse plate dimension	$F5$ (in)	1.375
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	1.25
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	3.125
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	243.8
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	182.8
Rupture demand/capacity ratio, $R_{u,fn} / \phi R_n$	D/C	0.88 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	56
External continuity plate dimension	C5 (in)	2.75
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	P_u (kips)	28.0
Cover plate overhang	C3 (in)	3.5
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625
Bridge plate factored tensile capacity, $0.9F_{yp}t_p([C3]-1")$	ϕR_n (kips)	70.3
Tension failure demand/capacity ratio, $P_u/\phi R_n$	D/C	0.40 OK

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe} + P_d)[C5]/(d_c + [C3])$	R_u (kips)	28.0
Weld 5 size	D_{W5} (1/16 in)	5
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W5}/16([C3]-1")(2\ sides)$	ϕR_n (kips)	52.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.54 OK

Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	360.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	23.9
Shear force from gravity	$V_{gravity}$ (kips)	7.0
Maximum probable moment at fuse location (see Step 2)	M_{pr} (K-in)	10000
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	66.1
Distance from alignment line to column	C1 (in)	6.375
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	17.6
Weld 7 size	D_{W7} (1/16 in)	3
Cover plate overhang	C3 (in)	3.5
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(0.707)D_{W7}/16[C3](2\ sides)$	ϕR_n (kips)	29.2
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.60 OK

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	66.1
Distance from alignment line to column	C1 (in)	6.375
Weld 6 demand, $V_u [C1]/d_b$	R_u (kips)	17.6
Weld 6 size	D_{W6} (1/16 in)	4
Cover plate overhang	C3 (in)	3.5
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	7
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{W6}/16(l_{eff})$	ϕR_n (kips)	58.5
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.30 OK

Shear plate shear failure check (when $W1 > 3/4"$ without shear plates, shear plates are used)

Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Weld 4 size	D_{W4} (1/16 in)	5
Weld 4 effective length, $d_c - 2 t_{fc}$	l_{eff} (in)	19.78
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16(l_{eff})$	R_u (kips)	137.7
Shear plate thickness, $t_{sp} \geq 1.1*W4\ or\ 3/8"$	t_{sp} (in)	0.375
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c - 2t_{fc})$	ϕR_n (kips)	222.5
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6 OK

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

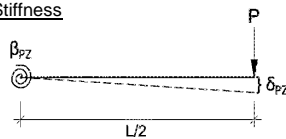
Show that effective panel zone stiffness $\geq 10 \times$ beam flexural stiffness

Effective Panel Zone Stiffness

$$K_{PZ} = P/\delta_{PZ}$$

$$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$$

$$K_{PZ} = \beta_{PZ}/(L/2)^2$$

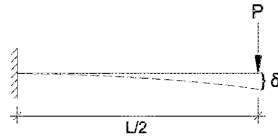


Beam Stiffness

$$K_B = P/\delta_B$$

$$\delta_B = P(L/2)^3/3EI$$

$$K_B = 3EI/(L/2)^3$$

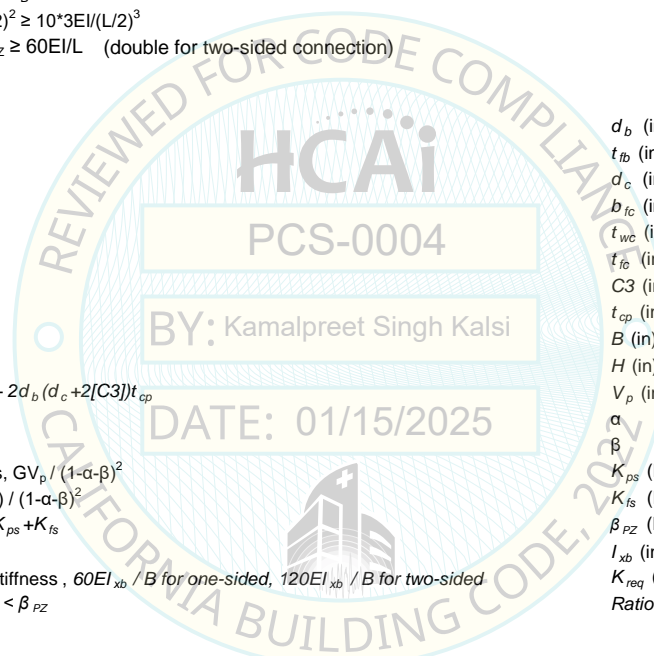


$$K_{PZ} \geq 10K_B$$

$$\beta_{PZ}/(L/2)^2 \geq 10 \times 3EI/(L/2)^3$$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	23.9
Beam flange thickness	t_{fb} (in)	0.68
Column depth	d_c (in)	21.7
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.6
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang	C3 (in)	3.5
Cover plate thickness	t_{cp} (in)	0.75
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])t_{cp}$	V_p (in ³)	1340.1
Column ratio, $(d_c - t_{fc})/B$	α	0.058
Beam ratio, $(d_b - t_{fb})/H$	β	0.108
Web and cover plate stiffness, $GV_p/(1-\alpha-\beta)^2$	K_{ps} (k-in)	2.15E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2)/(1-\alpha-\beta)^2$	K_{fs} (k-in)	1.43E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.17E+07
Beam moment of inertia	I_{xb} (in ³)	2100
Required panel zone spring stiffness, $60EI_{xb}/B$ for one-sided, $120EI_{xb}/B$ for two-sided	K_{req} (k-in)	2.03E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	Ratio < 1	0.94 OK



DuraFuse Frames® Calculations
HCAI CALCULATIONS
Connection ID DHF101
HSS10X10X3/4 Column and W14X38 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	<i>E</i> (ksi)	29000
Shear modulus (AISC 360-22)	<i>G</i> (ksi)	11200

Beams

Beam material grade	<i>Gr.</i>	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yb}</i> (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{ub}</i> (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yb}</i>	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tb}</i>	1.1

Columns

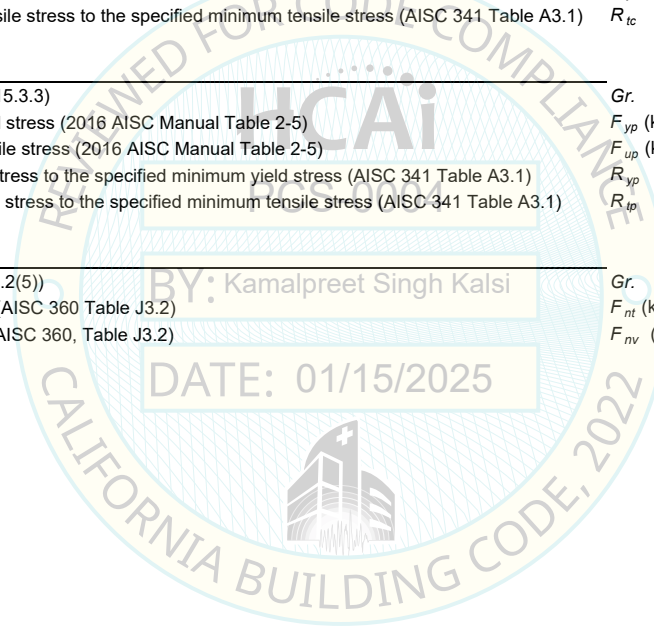
Column material grade	<i>Gr.</i>	ASTM A500 Gr.C
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	<i>F_{yc}</i> (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	<i>F_{uc}</i> (ksi)	62
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yc}</i>	1.3
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tc}</i>	1.2

Plates

Plate material (AISC 358-22 15.3.3)	<i>Gr.</i>	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	<i>F_{yp}</i> (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	<i>F_{up}</i> (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	<i>R_{yp}</i>	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	<i>R_{tp}</i>	1.2

Bolts

Bolt grade (AISC 358-22 15.5.2(5))	<i>Gr.</i>	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	<i>F_{nt}</i> (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	<i>F_{nv}</i> (ksi)	84



Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied
Beam limitation checks

Beam size		W14X38
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40
Beam depth	d_b (in.)	14.1 OK
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	397
Beam weight	W_b (lb/ft)	38 OK
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6
Beam web width-to-thickness ratio	λ_{web}	39.6 OK
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), $0.38(E/F_{yb})^{0.5}$	$\lambda_{p,flange}$	9.2
Beam flange width-to-thickness ratio	λ_{flange}	6.57 OK
Beam lateral bracing limit (AISC 358 15.3.1(5))	See bracing calculations	

Column limitation checks

Column size		HSS10X10X3/4
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36
Column depth	d_c (in.)	10
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit
Column weight	W_c (lb/ft)	89.5 OK
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit
Column flange thickness	t_{cf} (in)	0.698 OK
Axial force on the column (provided by EOR)	P_u (kips)	123.5
Gross area of the column section	A_g (in ²)	24.7
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.1
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	52.8
Column web width-to-thickness ratio	λ_{web}	11.3 OK
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	13.73
Column flange width-to-thickness ratio	λ_{flange}	11.3 OK

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in ³)	84.7
Column design axial load	P_u (kips)	123.5
Column area	A_{gc} (in ²)	24.7
Story Height	H (in)	180.0
Column depth	d_c (in)	10
Beam depth	d_b (in)	14.1
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM_{pc}^* (k-in)	8.86E+03
Beam plastic section modulus	Z_b (in ³)	61.5
Minimum bay width	B (in)	240.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	3075
Clear distance between column faces, $B - d_c$	L_h (in)	230.0
Live load factor in seismic load combination	f_l	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f_l L + 0.2S$	V_g (kips)	11.0
Beam shear (AISC 358 15.4a), $2M_{pr} / L_h + V_g$	V_b (kips)	37.7
Increase in moment from beam shear, $V_b (d_c/2)$	M_{uv} (k-in)	189
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma (M_{pr} + M_{uv})$	ΣM_{pb}^* (k-in)	3.26E+03
Column-beam moment ratio, $\Sigma M_{pc}^* / \Sigma M_{pb}^*$	Ratio > 1.0	2.71 OK

*Not applicable for top stories, see AISC 341 E3.4a(a1)

Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}

Beam moment demand from load combinations (from EOR)	M_u (k-in)	0
Beam plastic section modulus	Z_b (in ³)	61.5
Beam plastic bending moment, $F_{yb} Z_b$	M_p (k-in)	3075
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} \leq M_p$	M_{pr} (k-in)	3075 OK
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.625
Beam depth	d_b (in)	14.1
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	104.4
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	104.4

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	104.4
Beam area	A_{gb} (in ²)	11.2
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	42.8
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	147.2
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	10
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.625
Beam depth	d_b (in)	14.1
Column depth	d_c (in)	10
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	190.2
<hr/>		
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhang dimension	$C3$ (in)	2
Column depth	d_c (in)	10
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	14
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	315.0
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	315.0
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.47 OK
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Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)		
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag condition	h_{cp} (in)	20.10
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	452.3
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	452.3
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.42 OK
<hr/>		
Cover plate thickness check (AISC 341 E3.6e.2)		
Beam depth	d_b (in)	14.1
Beam flange thickness	t_{fb} (in)	0.515
Column flange thickness	t_{fc} (in)	0.698
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	13.07
Panel zone horizontal dimension, $d_c - 2t_{fc}$	w_z (in)	8.604
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.241
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	3.11 OK

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	0.75
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole, std}$ (in)	0.8125
Beam plastic section modulus	Z_b (in ³)	61.5
Beam flange thickness	t_{fb} (in)	0.515
Bolt hole area in section, $(d_{hole, std} + 1/8)t_{fb}$	$A_{bolthole}$ (in ²)	0.483
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in ³)	13.1
Beam plastic section modulus of the net section, $Z_b - Z_{boltholes}$	$Z_{b, net}$	48.4
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	3383
Beam fracture moment, $Z_{b, net} R_{tb} F_{ub}$	M_{fr} (k-in)	3459
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.98 OK

Step 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb} F_y$)	$P_{d, total}$ (kips)	85.5
Drag force per line of bolts, $P_{d, total}/2$	P_d (kips)	42.8
Bolt diameter	d_{bolt} (in)	0.75
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.442
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	37.1
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi r_n$	$n_{b, req}$	4.0
Number of bolts per line (AISC 358 15.2(1))	n_b	6 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.0625
Bolt spacing (see schedule)	s (in)	2.75
Plate thickness, t_p (see T2 in drawings)	t_p (in)	0.625
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	1.785
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a] + 3in - 2e_d] / s + 1$	$n_{p, min}$	2.6
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3 OK
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	3

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	147.2
Beam flange thickness	t_{fb} (in)	0.515
Fuse and top plate thickness	$T2$ (in)	0.625
Governing material thickness, min $(t_{fb}, T2)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	361.5
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	0.9375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.8
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	364.0
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o / 2$	l_{c2} (in)	0.6
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	23.9
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	387.9
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	222.7
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	222.7
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	167.0
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.88 OK

Bolt slip for wind load check

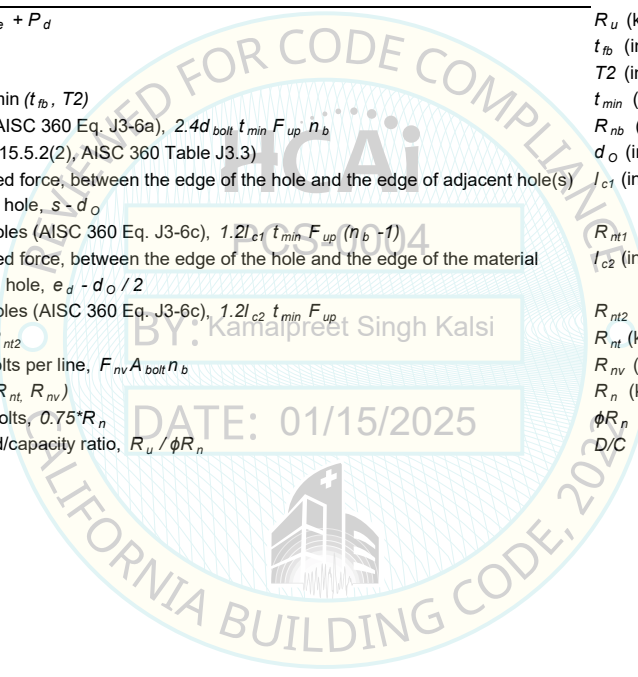
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	600
Bolt group demand from wind per line, $M_u / (2(d_b + t_p))$	R_u (kip)	20.4
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	35
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	6
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	71.2
Total factored bolt slip resistance, $0.85 R_n$	ϕR_n (kips)	60.5
Wind slip demand/capacity ratio, $R_u / \phi R_n$	D/C	0.34 OK

Step 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	104.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0
Bolt diameter	d_{bolt} (in)	0.75
Area of single bolt, $\pi \cdot d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.442
Bolt shear strength, $\phi F_{nv} A_b$	ϕR_n (kips)	37.1
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d) / \phi R_n$	$n_{b, req}$	2.8
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b, Bot}$	5 OK
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.0625
Bolt spacing (see schedule)	s (in)	2.75
Plate thickness, t_p (see T2 in drawings)	t_p (in)	0.625
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe} / [2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	1.785
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a] + 3in - 2e_d] / s + 1$	$n_{p,min}$	2.6
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p, Bot}$	3 OK
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	$n_{m, Bot}$	2

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	104.4
Beam flange thickness	t_{fb} (in)	0.515
Fuse and top plate thickness	$T2$ (in)	0.625
Governing material thickness, $\min(t_{fb}, T2)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} \Omega_b$	R_{nb} (kips)	361.5
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	0.9375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_o$	l_{c1} (in)	1.8
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	291.2
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_o / 2$	l_{c2} (in)	0.6
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	23.9
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	315.1
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	185.6
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	185.6
Factored shear capacity of all bolts, $0.75 \cdot R_n$	ϕR_n (kips)	139.2
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.75 OK



Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	14.1
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.0625
Beam flange width factor, 0 if $b_{fb} < b_{fc}$, 1.0 otherwise	α	0
Cover plate overhang dimension	C3 (in)	2
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max}d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	1.9085
Distance from alignment line to column	C1 (in)	5.125 OK

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per weld, $P_{d,total}/2$	P_d (kips)	42.75
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav-ortho}$ (kips)	10
Column depth	d_c (in)	10
Column flange thickness	t_{fc} (in)	0.698
Beam depth	d_b (in)	14.1
Plate thickness (see T2 on drawings)	t_p (in)	0.625
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	C6 (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	20.1
Gage dimension from the cover plate to the line of bolts on the external continuity plates	C5 (in)	2.375
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	19.10
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.38
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	0
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	8.604
Horizontal distance from center of rotation to geometric center	e_x (in)	0.40
Vertical distance from center of rotation to geometric center	e_y (in)	0.77

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, ($\Sigma V_{fe} + P_d$)*L ΣR_L (k-in) 3792

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	11.13	6.04	65.55	67.3
W4TL	11.51	0.00	23.34	0.0
W1LB	10.14	5.91	57.86	60.0
W4BR	9.79	0.00	22.65	0.0
W1RC	4.70	3.64	11.78	17.1
W4TC	10.58	0.00	2.60	0.0
W1LC	5.48	3.71	10.09	20.3
W4BC	9.04	0.00	3.05	0.0
		6.04	65.55	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	6.0
Orientation of Resultant Force	θ_r (degrees)	65.6
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.38
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma R_L)$	r_{uv} (kip/in)	7.7
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c * l_{We1})$	r_{un} (kip/in)	8.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	11.1
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.25
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	11.87
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.94 OK

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.75
Column depth	d_c (in)	10
Column flange thickness	t_{fc} (in)	0.698
Cover plate overhang dimension	C3 (in)	2
External continuity plate dimension	C5 (in)	2.375
Weld 2 size	D_{W2} (1/16 in)	6
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.38
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	3.07
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe} + P_d) / (d_c + 2[C3])$	r_{uv2} (kip/in)	10.5
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff} (d_c + 2[C3]) - 2l_{hb2} - l_{we2,eff})$	r_{un2} (kip/in)	11.2
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	15.3
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2 \text{ sides})$	ϕr_n (kip/in)	16.7
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	D/C	0.92 OK

Failure of external continuity plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	104.4
External continuity plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	349.5
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	147.2
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c + 2[C3])t_p$	ϕR_n (kips)	262.5
External continuity plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c + 2[C3])/2)^2$	ϕM_n (kips)	1378.1
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.81 OK

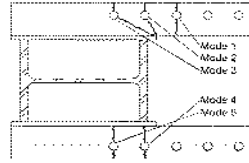
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8
Column fillet dimension	k (in)	1.0
Column fillet dimension	$k1$ (in)	0.7
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3] - 2l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	42.3
Column flange width	b_{fc} (in)	10
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc} - 2k1)$	$l_{w2,t}$ (in)	37.216
Weld 2 size	D_{W2} (1/16 in)	6
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	471.2
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	414.4
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	1022.2
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	766.6
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.19 OK

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC Specification Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	73.6
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate width	$C4$ (in)	4.125
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125
Net area of external continuity plate cross section, $t_p ([C4] - (d_{hole,std} + 1/16))$	A_n (in ²)	2.031
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	132.0
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	99.0
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.74 OK



Mode 2: Rupture through the first bolt hole in the direction of the column

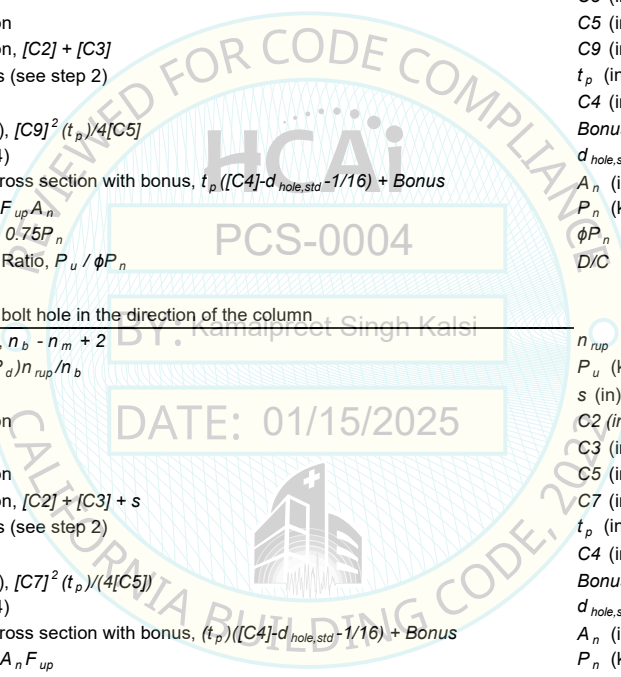
Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	98.1
External continuity plate dimension	$C2$ (in)	-1.5
Cover plate overhang dimension	$C3$ (in)	2
External continuity plate dimension	$C5$ (in)	2.375
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	0.5
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate width	$C4$ (in)	4.125
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p)/4[C5]$	$Bonus$ (in)	0.016
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125
Area of external continuity plate cross section with bonus, $t_p ([C4] - d_{hole,std} - 1/16) + Bonus$	A_n (in ²)	2.048
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	133.1
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	99.8
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.98 OK

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	122.6
Bolt spacing (see schedule)	s (in)	2.75
External continuity plate dimension	$C2$ (in)	-1.5
Cover plate overhang dimension	$C3$ (in)	2
External continuity plate dimension	$C5$ (in)	2.375
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	3.25
External continuity plate thickness (see step 2)	t_p (in)	0.625
External continuity plate width	$C4$ (in)	4.125
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p)/(4[C5])$	$Bonus$ (in)	0.695
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125
Area of external continuity plate cross section with bonus, $(t_p)/([C4] - d_{hole,std} - 1/16) + Bonus$	A_n (in ²)	2.726
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	177.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	132.9
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.92 OK

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	98.1
External continuity plate dimension	$C2$ (in)	-1.5
Cover plate overhang dimension	$C3$ (in)	2
External continuity plate width	$C4$ (in)	4.125
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	0.5
External continuity plate thickness (see step 2)	t_p (in)	0.625
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	2.578
Gross shear area, $t_p [C9]$	A_{gv} (in ²)	0.313
Net tension area, $A_{gt} - ((t_p) * (d_{hole,std} + 1/16))$	A_{nt} (in ²)	2.031
Net shear area, A_{gv}	A_{nv} (in ²)	0.313
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	141.4
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	106.1
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.93 OK



Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 5 rupture demand, $(V_{fe} + P_d) * n_{rup} / n_b$	R_u (kips)	122.6
Bolt spacing (see schedule)	s (in)	2.75
External continuity plate dimension	C2 (in)	-1.5
Cover plate overhang dimension	C3 (in)	2
External continuity plate width	C4 (in)	4.125
External continuity plate dimension, $[C2] + [C3] + s$	C7 (in)	3.25
External continuity plate thickness (see step 2)	t_p (in)	0.625
Diameter of bolt holes (see step 4)	$d_{hole, std}$ (in)	0.8125
Gross tension area, $t_p [C4]$	A_{gt} (in ²)	2.578
Gross shear area, $t_p [C7]$	A_{gv} (in ²)	2.031
Net tension area, $A_{gt} - (t_p)(d_{hole} + 1/16)$	A_{nt} (in ²)	2.031
Net shear area, A_{gv}	A_{nv} (in ²)	2.031
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	193.0
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	144.7
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.85 OK

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Minimum bay width	B (in)	240.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	11.0
Maximum probable moment at fuse location (see Step 2)	M_{pr} (k-in)	3075
Beam shear demand, $(2M_{pr} / (B - d_c)) + V_{gravity}$	V_u (kips)	37.7
Beam depth	d_b (in)	14.1
Beam web thickness	t_{wb} (in)	0.31
Beam web area, $d_b t_{wb}$	A_w (in)	4.371
Beam web width-to-thickness ratio, h / t_w	I_{web}	39.6
Beam web shear buckling coefficient (AISC 360 G2.1)	K_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_wC_v$	ϕV_n (kips)	131.1
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.29 OK

Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2
 C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d, total}$ (kips)	85.5
Drag force per line of bolts, $P_{d, total} / 2$	P_d (kips)	42.8
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	294.3
Beam flange thickness	t_{fb} (in)	0.515
Beam flange width	b_{fb} (in)	6.77
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole, std}$ (in)	0.8125
Number of holes along each bolt line (see step 5)	n_b	6
Length of connection on beam	B4 (in)	14.8125
Distance from beam centerline to bolt line	P2 (in)	2.375
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in ²)	1.040
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in ²)	15.257
Net tension area, $A_{gt} - (t_{fb})(d_{hole, std} + 1/16)$	A_{nt} (in ²)	0.590
Net shear area, $A_{gv} - (t_{fb})(d_{hole, std} + 1/16)(2n_b - 1)$	A_{nv} (in ²)	10.300
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	440.0
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	330.0
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89 OK

Step 12. Determine the required number of shear tab bolts for bolt shear

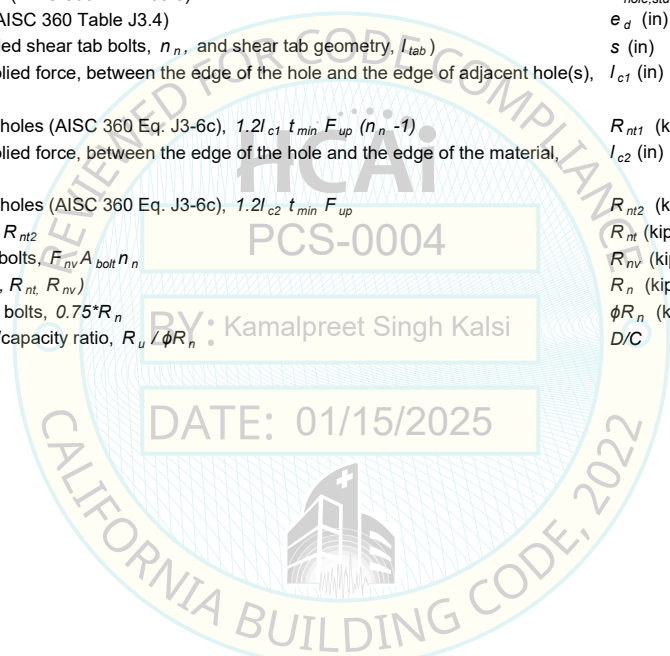
The required strength of shear tab bolt group (see step 10)	R_u (kips)	37.7
Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Area of single bolt, $\pi d_{bolt}^2 / 4$	A_{bolt} (in ²)	0.442
Nominal shear capacity of a single bolt, $F_{nv} A_{bolt}$	r_n (kips)	37.1
Factored shear capacity of a single bolt, $0.75 r_n$	ϕr_n (kips)	27.8
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	1
Provided number of shear tab bolts	n_n	3 OK

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	11.625
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	10.625
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	10.625 OK

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.31
Shear tab thickness	$T3$ (in)	0.375
Governing material thickness, $\min(t_{wb}, T3)$	t_{min} (in)	0.3
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_n$	R_{nb} (kips)	108.8
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	0.8125
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.000
Bolt spacing (based on provided shear tab bolts, n_n , and shear tab geometry, l_{tab})	s (in)	4.313
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	3.5
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_n - 1)$	R_{nt1} (kips)	169.3
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.6
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	14.4
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	183.6
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	111.3
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	108.8
Factored shear capacity of all bolts, $0.75 R_n$	ϕR_n (kips)	81.6
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.46 OK



Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	35
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b\mu D_u h_f n_s$	F_b (kips)	17.8
Provided number of shear tab bolts (see step 12)	n_n	3
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	53.4
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	37.7
Distance from alignment line to column	$C1$ (in)	5.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	193.4

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Slotted bolt hole width (standard slot width)	w_b (in)	0.8125
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.375
Shear tab length (see step 12)	l_{tab} (in)	10.625
Beam T dimension limit	T_{beam} (in)	11.625
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	4.1875
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in ²)	3.98
Shear tab net area, $A_{gv} - n_n (w_b + 1/16)t_{tab}$	A_{nv} (in ²)	3.00
Effective net area, $A_e = A_{nv}$	A_e (in ²)	3.00
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up}A_{nv}$	ϕR_{nv} (kips)	87.75
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up}A_e$	ϕR_{tn} (kips)	146.25
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{tn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.17 OK

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	37.74
Required axial strength of the shear tab (see above)	P_u (kips)	53.39
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	193.41
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in ²)	3.98
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in ³)	10.58
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp}A_{gv}$	ϕR_{ny} (kips)	119.53
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp}A_{gv}$	ϕR_{nt} (kips)	179.30
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	476.26
Shear tab yielding demand/capacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nt})^2 + (V_u/\phi R_{ny})^4$	D/C	0.50 OK

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	10.625
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	37.7
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	3.6
Number of slotted bolt holes in shear tab	n_{vb}	3
Slotted bolt hole spacing	s_{slot} (in)	4.1875
Distance from line of bolts to shear tab weld	$C1$ (in)	5.125
Size of weld in 1/16 ^{ths} of an inch	D_{w3}	7
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	193.4
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u) / l_{w3}^2 - 2F_b n_{vb} / l_{w3}$ except when bridge plates present, $F_b n_{vb} / l_{w3}$	r_{un} (k/in)	18.1
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	18.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{w3}/16(1")(2 \text{ sides})$	ϕr_n (kips)	19.5
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.94 OK

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.0625
Beam depth	d_b (in)	14.1
Length of the shear tab (see step 12)	l_{tab} (in)	10.625
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.053
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.125 OK
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.125

Step 15. Check the top plate for shear yielding and shear rupture (AISC Specification Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	147.2

Yielding check

Top plate thickness (see step 2)	t_p (in)	0.625
Top plate dimension (see schedule)	$P6$ (in)	17.25
Gross shear area of top plate, $(P6)(t_p)$	A_g (in ²)	10.781
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	323.4
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	323.4
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.46 OK

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1
Number of short slotted bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g - (l_s n_b)$	A_n (in ²)	7.031
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	274.2
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	205.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.72 OK



Fig. C-15.8. Shear rupture of net section on top plate.

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8
Total number of bolts (see step 5)	n_b	6
Number of bolts in the M region (see step 5)	n_m	3
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d), P_u$ (kips)		73.6

Yielding check

Top plate dimension	$P4$ (in)	1.375
Top plate dimension	$P5$ (in)	2.125
Top plate thickness (see step 2)	t_p (in)	0.625
Gross area, $([P4]+[P5])*t_p$	A_g (in ²)	2.188
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	109.4
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	98.4
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.75 OK

Rupture check

Top plate dimension	$P4$ (in)	1.375
Top plate dimension	$P5$ (in)	2.125
Top plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	0.9375
Effective area of rupture, $([P4]+[P5]-d_o-1/16)t_p$	A_e (in ²)	1.563
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	101.6
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	76.2
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.97 OK

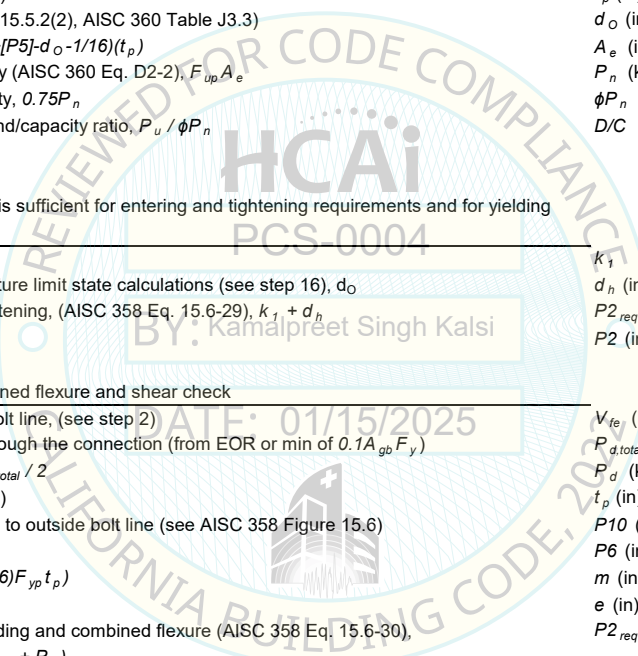
Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Entering and tightening check

Beam flange fillet dimension	k_1	0.8
Bolt hole diameter used for rupture limit state calculations (see step 16), d_o	d_h (in)	0.9375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	1.75
Provided P2 dimension	$P2$ (in)	2.375 OK

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8
Top plate thickness (see step 2)	t_p (in)	0.625
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	8.125
Inside length of the top plate	$P6$ (in)	17.25
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	8.721
Dimension, $([P6] - m) / 2$	e (in)	4.265
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-2.5
Provided P2 dimension	$P2$ (in)	2.375 OK



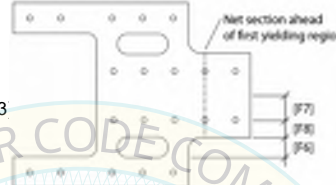
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC Specification Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	1.625 OK
Top plate thickness (see step 2)	t_p (in)	0.625
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.6 OK

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

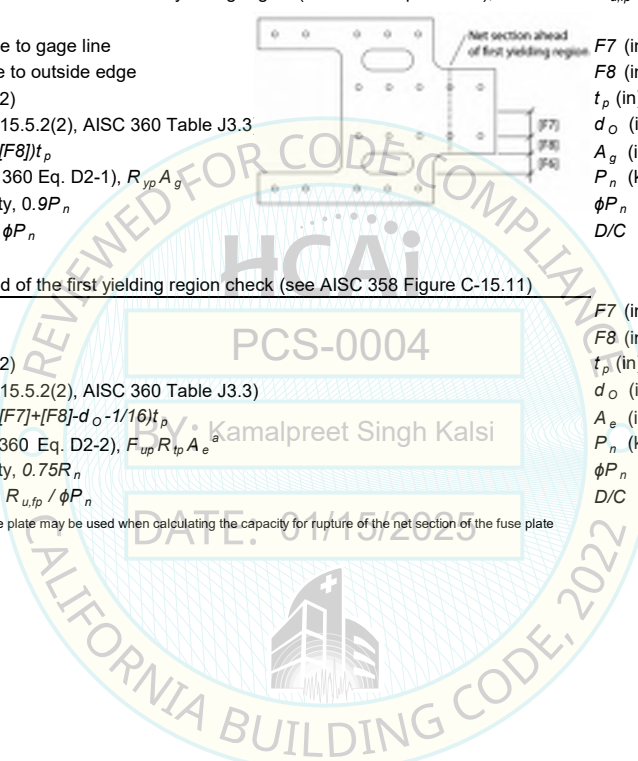
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	$n_{b, Bot}$	5
Number of bolts in the M region (see step 5)	$n_{m, Bot}$	2
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	83.5
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.375
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.75
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	0.9375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in ²)	6.406
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	320.3
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	288.3
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.29 OK



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.375
Fuse plate dimension	$F8$ (in)	2.75
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	0.938
Effective net area of rupture, $2([F7]+[F8]-d_o-1/16)t_p$	A_e (in ²)	5.156
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	P_n (kips)	402.2
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	301.6
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.28 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate



Step 19. Determine the yielding region depth [F2]
Applicable to AISC 358-22

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	104.4
Fuse plate thickness(see step 2)	t_p (in)	0.625
Width of fuse plate cut-out	$F6$ (in)	1.625
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6])/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	1.521
Provided yielding region depth	$F2$ (in)	1.500 OK
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	56.3
Ratio	V_{fe} / V_y	1.9
Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 \leq [F6] / [F2] \leq 1.25$	$F6 / F2$	1.08 OK

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC Specification Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ (kips)	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	$n_{b, Bot}$	5
Number of bolts in the M region	$n_{m, Bot}$	2
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fn}$ (kips)	83.5
Yielding check		
Fuse plate dimension	$F4$ (in)	1.375
Fuse plate dimension	$F5$ (in)	1.125
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	0.9375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in ²)	3.125
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	156.25
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	140.63
Yielding demand/capacity ratio, $R_{u,fn} / \phi R_n$	D/C	1.00 OK
Rupture check		
Fuse plate dimension	$F4$ (in)	1.375
Fuse plate dimension	$F5$ (in)	1.125
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_o (in)	0.9375
Effective net area of rupture, $2([F4]+[F5]-d_o-1/16)(t_p)$	A_e (in ²)	1.875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e^a$	R_n (kips)	146.3
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	109.7
Rupture demand/capacity ratio, $R_{u,fn} / \phi R_n$	D/C	0.76 OK

^aThe expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate